INVESTIGATION OF THE ACTUAL STRESSES IN STIRRUPS OF REINFORCED CONCRETE T-BEAMS

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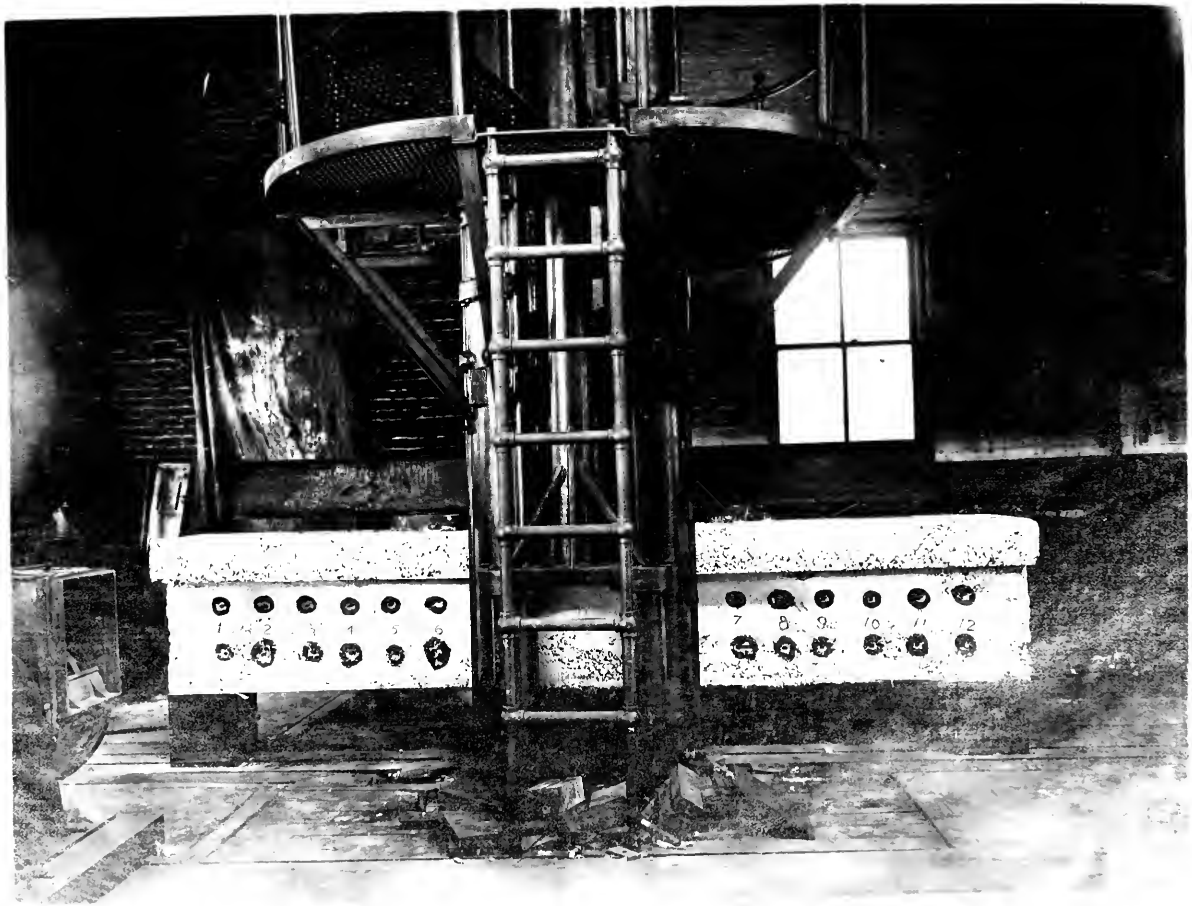
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FOREWORD.

The exceedingly rapid developement of reinforced concrete construction during the last twenty-five years has made necessary the investigations of the reinforced concrete beams, and at the present time the stresses taken by the steel in tension and the concrete in compression are well understood. The results have been formulated satisfactorily, and are being used in all reinforced concrete designing.

Soon after the use of reinforced concrete became popular, failure due to what is known as diagonal tension became frequent. To avoid this, stirrups or turned up horizontal bars were used. The stresses taken by this steel were calculated theoretically, and at present are so calculated on the assumption that the steel takes three fourths or two thirds of the total vertical shearing stress.

Several tests made on reinforced concrete at various experiment stations show intentions at the outset of determining the actual stresses in the steel, but in the conclusion merely the remark is made that probably the steel takes two thirds or three fourths of the total stress.

The authors tested the actual stresses in the stirrups and inclined bars. For this purpose three reinforced concrete beams, twelve feet long, and weighing four thousand pounds each were used. Two of these beams were designed with stirrups for web reinforcing, and the remaining beam was reinforced with inclined bars. The design was so made that the beam would fail just by diagonal tension and not by flexure.

The entire work including the theoretical design, the construction of forms, mixing and placing of concrete and testing the beams was performed by the authors.

In the pursuance of this work the authors wish to express their indebtedness to the Civil Engineering Department for the use of materials; the Mechanical Engineering Department for the use of its laboratories; Professors Huntly and Dean for suggestions in carrying out the work; Professor J. E. Snow for photographic work; Mrs. Julia Beveridge for the preparation of the bibliography; Messrs. Stern, Taylor and Smith in placing the concrete.

The tables for design were those found in Hool Volume 1.

Armour Institute,
May 31, 1918.

Jesse Nitka
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Leslie Weiss.

A Short Review of Reinforced Concrete.

Although the present boom in the reinforced concrete industry is tremendous it must not be supposed that this is something new. The real rapid evolution in the last twenty-five years has been due to the discovery of the economic production of Portland Cement. This justifies a retrospect of the use of cement.

The very earliest habitations of man show traces of cementing. Egyptian history mentions the use of a substance, which we now call cement, in the construction of the pyramids. Pliny mentions that the columns which adorn the peristyle of the Egyptian labyrinth were made of a material which we would now call concrete. The durability of this material is obvious since these columns which are three thousand six hundred years old are still standing in good condition. The dome of the Pantheon erected at the beginning of the Christian era was

constructed of a similar material. The Romans used a concrete consisting of lime, stone and sand many years before Christ in the construction of walls, aqueducts, and piers. Some of the old Roman Military roads still show the rugged concrete construction of the early days.

They used puzzolana or volcanic ash to make the lime hydraulic. For a mortar they used lime and powdered brick. Vitruvius writing in the first century describes the method of making concrete with lime along. The formula given is:

12 parts well pulverized puzzolana
6 parts quartz well washed
9 parts of rich lime
6 parts broken stone.

This was called "opus caementum" from which we derive our word cement. In the middle ages concrete was used for construction of walls and foundations. Generally it was used for coring purposes.

The year 1750, perhaps may be considered

as a land mark in the cement industry; for it was in that year that John Smeaton discovered in the construction of the Eddystone Light-house the fact that certain limes became hy-draulic. The next century Vicat became world famous for his patents in the industry. The natural cement manufacture was a result of the development of the early Roman idea, by an Englishman named James Parker. He patented a product which resulted from the calcination and grinding of nodules of argillaceous limestone; this Parker called Roman cement.

The real beginning of what is now called the Portland cement industry may be attributed to Joseph Aspdin of Leeds, England. In 1824 while supervising the construction of a canal he found that an artificial mixture of slaked lime, and clay, highly calcined formed a hydraulic product. This he called Portland cement after the Portland stone which he used. But somehow or other the industry was not developed until about twenty

years later when J. B. White and Sons in Kent, England commenced its manufacture. This brought about enthusiastic experimenting by the Engineers of England and France. It must be said here that this industry, in its early development, made more rapid advances in France than in any other country. The use of Beton by the French in sheltering a mole in the harbor of Algiers in the year 1833 showed that they also were experimenting. Beton is a product which will be more fully described later in this treatise. In 1866-71 Mr. John Grant made a report of a series of experiments upon the practical action of cements, mortars, and concretes under varied conditions. This report was made to the Institution of Civil Engineers.

The report of 1869 of the American Commission to the International Exposition in France has some very interesting notes in regard to the concrete industry as exhibited. It is stated that: "one of the most interesting

displays was made by the French Cement Company Boulogn-sur Mer which received a gold medal. An apparatus for testing the strengths of cements was included in their show. This cement was artificially repared by mixing intimately with care 79-1/2% of powdered carbonate of lime, 20-1/2% of clay and burning at a high temperature. The Vicat cement is much more used in France now than formerly. It is manufactured by Mr. J. Vicat a graduate of the Ecole Politechnique. He is the son of the celebrated Engineer." This report shows that it was about at this time that a greater confidence was placed in the use of cement and concrete. The Beton Coignet which was so popular in France at this time is a conglomeration of sand, pebbles, broken stone, lime and water. It was used to build warehouses, churches, walls, foundations, granaries, and cellars.

Later sewers, aqueducts, waterpipes, cisterns, and reservoirs also were made of this

material. The mixing machines for this were not so unlike the ones we use at the present time. The forms also were of the present type in use. Beton Coignet which was at that time patented by Mr. Coignet was first experimented upon by him in 1858, in the construction of the Socoa breakwater at St. Jean. All this time no mention of the use of reinforcing was made. A little incident which later resulted in the development of the construction of the reinforced concrete Arch bridges, which should be mentioned here is the construction of concrete water basins, by J. Monier in 1868. Mr. Monier used wire netting with his concrete so that he could build thinner slabs than could be built with the plain concrete.

England soon followed in a more practical way when the patents of Hennebique and Williams established a standard method of reinforced concrete building construction. In 1888 Ransome patented a method of reinforced construction.

In 1890 the first tests of the Monier Arch were made and found satisfactory although no definite analysis of the stresses were made or formulas published. This test was made upon a reinforced concrete arch R.R. bridge located in Malzendorf, Austria and built for the Southern R.R. Company of Austria. The reinforcing consisted of wire netting composed of wire .39 and .28 inches in diameter and spaced 3-1/4 inches center to center. The concrete was 1-4-6 mixture. The total length of the bridge was 32.8 feet. The results of practical tests proved the bridge to be perfectly safe. The next year a little of this type of construction found its way to the United States although American engineers at first tried to avoid it. In the month of June 1891 Mr. Phineas Ball was reported to have constructed a portland cement floor between the front of his residence and his sidewalk. This he stiffened by one half inch

iron rods placed lengthwise three inches below the surface and spaced nine inches apart.

Although considerable advance had been made, by 1893, in the construction of the Monier system, statements were made in notable engineering publications that reinforced concrete construction would make little progress in the future. The advances made by the Monier Company were such that long span arches were now being constructed. Also the calculation of the stresses and a satisfactory application of the same were at this time brought forth. Formulas for the strength of concrete were given in 1893 by Mr. A. F. Bruce, in an article in the Engineering News.

Some of the disadvantages forwarded by some engineers who had no faith in this type of construction may be summed up as follows: the wire would eventually become corroded by coming in contact with the moist concrete; iron and concrete would not bond well, nor act uniformly;

iron wire would cause cracking due to unequal expansion of the iron and concrete. The Monier Company made tests which refuted each one of these objections, as the reader will no doubt note that all these difficulties are very minor in the present day type construction.

Soon after this the first concrete arch bridge was constructed in the United States. It was built to carry Pine Road over Pennypack Creek, in Philadelphia. This structure was designed by Mr. C. A. Frik. It is 25-3/4 feet long and is reinforced one half inch wire mesh placed two inches apart.

The year 1893 marks the beginning of experimentation upon cement, concrete, and steel to determine how the properties of each effect the other and how they may be combined advantageously. The various engineering societies and experiment stations now took up the matter of investigation of reinforcing concrete and may different tests were conducted,

the maximum work being done along this line during the years 1903-1905. As a result of these investigations analysis of reinforced concrete had been made and the stresses taken by the steel in tension and by the concrete in compression no longer remained a mystery, but the results of these analyses were used in designing structures which are at present standing well the practical tests. In the last few years the establishment of a Concrete Institute has made it possible to standardize methods of design.

It must be mentioned here that whereas concrete may be subjected to large compressive stresses it is utterly useless when subjected to tension. Hence the first problem which presented itself was that of using steel reinforcing where~~e~~ver tensile stresses are likely to be present. Practice showed that even reinforcing plentifully for the tension with steel still subjected the structure to

failure by cracks formed diagonally across the surface of the wall or beam. Analysis of this action has developed the theory that this failure is caused by a stress which is the result of the horizontal and vertical shear which is always present in a loaded member. This stress is called diagonal tension. In an attempt to neutralize the effects of this web stress, stirrups and bars bent up at an angle have been used with some success, but not with the complete abolition of these cracks. The reinforcing in the web is calculated entirely theoretically and is based upon no practically data collected in regard to the web stresses. It is surprising that after the numerous experiments were performed in the last few years none show what proportion of the stress is actually taken by the web reinforcing. It is the object of the authors of this work to design according to the present day standard practice reinforced concrete beams with web re-

inforcing to neutralize the effects of the web stresses. The web reinforcing was designed upon the assumption that the steel in the web takes two thirds of the total direct shearing stress. The actual stresses in this reinforcement were then measured as described later in the treatise. The experimenters wish merely to determine the relation of the actual stresses in this reinforcement to the theoretically calculated stresses.

General Considerations

Consider a beam of a homogeneous material supported at both ends and for example loaded at the center with a single load although it might be loaded in various ways; but for the purpose at hand a single load at the center will do. Analyzing the stresses in such a beam will show that at the bottom the (plane A-A, Fig. 1) fibers of the material tend to pull apart, and at the top these fibers are pushed together.

In the terms of mechanics such action as occurs in the plane A-A is called tension, and such as in B-B is termed compression. The tendency for the beam to separate in a vertical plane due to the force P is called the vertical shear.

The properties of concrete are such that it can stand rather high compressive stresses, but it is practically useless when subjected to

tension. Therefore, to use concrete in buildings and other structures where it is under the influence of tension, the latter must be taken care of by some other material which will resist this stress. Steel has been found to be very adequate for this purpose since it has a large tensile strength. Thus generally steel is used for reinforcing in the bottom of a reinforced concrete beam. In this way the maximum strength may be obtained at a minimum cost. The stress taken by the steel is generally transmitted to the concrete by the bond between the steel and concrete. This bond stress is only one of the elements which generally may be included in the web stresses.

It has been found that with this type of reinforcing when a beam is subjected to loading, at a certain time failure occurs due to diagonal cracks formed as shown in Fig. 3. Careful analysis of the action occurring in such a beam explains this action by attributing

it to web stresses.

This web stress may include the bond stress, shearing stress in various directions, and tensile and compressive stresses in a direction other than that parallel to the axis of the beam. Taking the above factors into consideration would make the calculation of these stresses tedious and complicated, thus for designing, certain assumptions must be made.

Consider this failure due to the combination of principle stresses, namely the horizontal and vertical shear. The combined effect is called diagonal tension, and it is the effects of this that the web reinforcing should overcome. When a beam is subjected to flexure there is a tendency for the upper longitudinal layers to slide past the lower if the beam is considered as being made of layers, this sliding force is what is known as horizontal shear. In addition to this also a vertical shear is present, which is that stress which

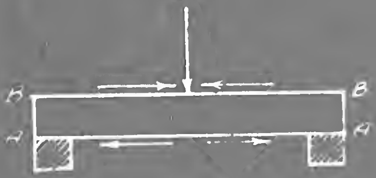


Fig 1.

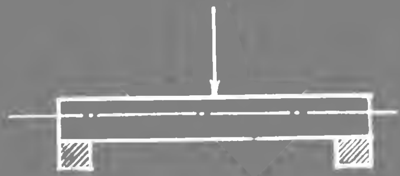


Fig. 2.

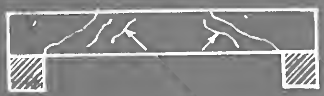


Fig. 3.

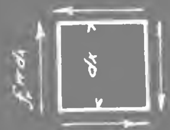


Fig. 4

Fig. 5

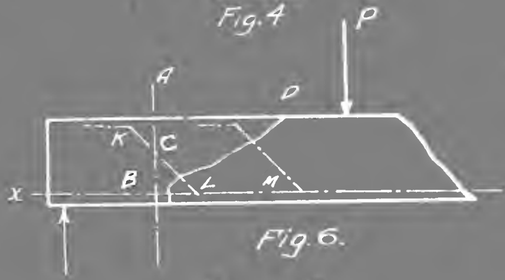


Fig. 6.

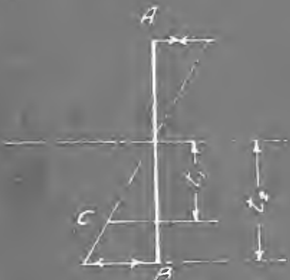


Fig. 7.

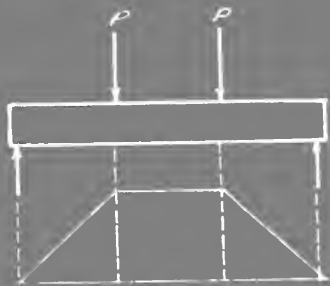


Fig. 8.

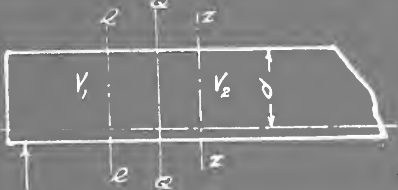


Fig. 9.

tends to separate the beam in a vertical plane at right angles to the neutral plane.

Let Fig. 4 represent a minute portion of the side of a beam, and the analysis will clearly show what action takes place. Call the sides x and the width of the beam w. As stated above two sets of shearing forces will act upon it, vertical and horizontal. These shears may be considered as forming couples in the vertical and horizontal planes. For a small distance dx the horizontal fiber stresses balance each other and may be neglected. Let the horizontal shear be represented by f_h and the vertical shear f_v, then if the forces acting on this particle be in equilibrium the moment must be equal and may be expressed:

$$(f_h \; w \; dx)dx = (f_v \; w \; dx)dx$$

$$f_h = f_v$$

Let a common expression v represent these values for shear.

The vertical shear at the top and bottom of the beam are zero with a maximum value at the neutral axis. Also it should be noted that the horizontal fiber stress is zero at the neutral azis and maximum at the top and bottom. Hence it may be seen that a maximum effect of diagonal tension will occur where the horizontal fiber stress is small compared with the vertical shear; this occurs near the supports.

References in mechanics show a combination of shear and tension produce a maximum diagonal tensile unit stress, the expression for which is written:

$$t = \frac{1}{2} s \pm \sqrt{\frac{1}{4} s^2 \pm v^2}$$

when t is the unit diagonal tensile stress, s the horizontal tensile unit stress in the concrete, and v is the horizontal n vertical shearing unit stress. This maximum diagonal tension makes an angle with the horizontal equal to $\tan 2\theta = \frac{2v}{s}$. If there is no tension

in the concrete $t = v$ and the maximum diagonal tension makes an angle of 45° with the horizontal and is equal in intensity to the vertical shearing stress at that point, and it is upon this result that the web reinforcing is designed for vertical shear.

This shows that the value of the diagonal tensile strength depends upon the tensile stress in a horizontal direction at a given point as well as upon the amount of the horizontal and vertical shearing stress developed. If $s = c$ and $t = v$ and the direction of t will be 45° from the horizontal.

The above considerations show that "T" is quite indeterminate and hence the common practice has become to design the steel in the web, for the vertical shear, although the actual diagonal tension is greater than the vertical shear.

Failure by diagonal tension generally starts in a crack at the bottom of the beam

which then extends horizontal to the reinforcement and then branches out somewhere above this running in a direction away from the support. Failure due to diagonal tension usually is sudden as in the case when concrete is tested in tension. When the reinforcing consists only of bars running straight through, part of the shear is carried forward by the stiffness of the rods and failure is slower and more progressive.

Bent Up Bars.

It has been stated that with the reinforcement running straight through, the flexural stress in it decreases from the loadcd point to the supports. If, however, the bars are bent up a different distribution of stresses takes place. The stress in the steel will now become greater than it was when in a horizontal position. This may be shown by the mathematical expression for the stress in such a bar. $S = \frac{M}{Ajd}$ for the stress in a horizontal

bar and for a turned up bar $S = \frac{M}{Ajd} \times \sec \phi$ when ϕ is the angle that the bar makes with the horizontal, (see Fig. 5.) Under ordinary assumptions the stress in the bar might be nearly constant to the end, but in a reinforced concrete beam the tensile strength of the concrete at the bottom of the beam operates to take the greater part of the tension which goes to make up the bending movement for sections toward the end of the beam, and besides, the variation from beam action near the end of the beam changes the stress distributions in sections near the end of the beam. If the tensile stress in the steel remained uniform to the end of the bar, the bond stress required at the end would be enormous. Since concrete has some tensile strength the above condition is impossible; high bond stresses must also exist at certain points. It becomes obvious now that a modification of the distribution of

the tensile stress, throughout the length of the reinforcing bar and of the bend between the steel and concrete at different cross sections must be accompanied with a change in distribution of shearing and diagonal stress. If the bend stress is small, the shearing stress will be small, and hence a low diagonal tension. Increasing the first two, however, will increase the last one with the bar bent in a diagonal part of the stress will be taken by it, but any vertical crack formed may form a diagonal crack along the line of reinforcing.

This last consideration was based upon the fact that all the bars were turned up at the same point. When part of the bars run through and others are turned up still another distribution of stresses takes place. For an explanation of this case let figure 6 aid as an illustration. Consider one half of the bars bent up along the line kl and remainder running on through to x at pm both sets of bars are at

the same elevation and have the same tensile stresses. Away from l toward the end of the beam the tensile stresses in kl will be less than in xl as the maximum bension occurs at the lowest fibers of the beam. If it may be considered that a plane section before bending remains so after bending then the deformation set up in the two bars may be represented graphically as shown in Fig. 7. Therefore, the tensile stresses being proportional to the deformation may be expressed

$$\frac{f_{t_1}}{f_{t_2}} = \frac{t_1}{t_2}$$

(Fig. 7) where f is the unit tensile stress in the bar the subscript of which it bears. This shows that the stress in the rod at B will take the greater part of the stress due to the bending movement, hence, there will be a large change in stress from l to c, hence a comparatively large bond strength must be developed. It will be higher than that

calculated by the formula. From c to k the bond, according to this analysis, will be less than the figured amount. In the lower bar as increments are taken in a direction toward the support, the decrease in stress will become more rapid, and hence a larger bond will be necessary. It must not be forgotten that this is all based upon the assumption that a plane section, before bending remains a plane section after bending, and it is evident that such a condition not probable in a concrete beam. This is another difficulty which enters into the determination of the web stresses.

Bending up the bars at l increases the vertical shear between l and c, the diagonal tension being taken by the inclined bar; at the same time less bond is developed between l and b, hence a lower diagonal tension than usual can be expected here. If bars were now turned up at b a similar analysis could be used, and this shows how difficult it would be

to theoretically analyze the mechanical action taking place.

As a result of the foregoing study it seems best to consider the diagonal rods to take the diagonal tension, provided they are of the proper area, and are properly distributed. Let s be the distance between bent up bars, v the total vertical shear at the point desired, and d be the distance from the compression face to the center of the tensile reinforcement, 1/j is the ratio of the distance from the center of gravity of the compressive stresses to the center of the steel to the distance from the compression face to center of the steel.
Then the diagonal stress taken by the reinforcement may be expressed as $.707 \frac{vs}{jd}$. One of the beams in this experimental work has been reinforced by inclined bars, the stresses of which will be determined by the extensometer measurements of the elongations of the bars. The relation of the actual and theoretical

stress is desired.

Determination of Points to Turn Up Rods.

The number of rods turned up should be such that the remaining rods will be sufficient to carry the tension due to bending. This merely resolves itself into determining what horizontal length of bar is necessary for a certain bending moment. To find this a moment diagram is plotted, and then the total number of bars distributed so as to sufficiently take care of the moment at different points in the beam. For instance consider a beam loaded as in Fig. 8.

On the line of maximum moment lay off the total number of rods required and draw horizontals through them; where they cut the moment curve is the place to turn up the rods as they supply enough length for the bending moment shown by the area cut off by the line on the moment diagram.

Web Reinforcement With Stirrups.

One of the most popular forms of web strengthening consists of vertical stirrups spaced close together. The vertical stirrup may be considered as taking the vertical component of the diagonal tension, which is the direct vertical shearing stress. Thus this method of strengthening takes care of the largest component of the stress.

These stirrups are "U" shaped or looped and enclose the longitudinal reinforcement, generally being fastened to it so as to act as a rigid member.

The standard method of the design of these stirrups is given in below..

LetVFig. 9 be the total shear and s the spacing of the stirrups. At a section RR the unit shear will equal $\frac{V_1}{jdb}$; thus at a section zz it will equal $\frac{V_2}{jdb}$; thus at a section gg it will equal $\frac{V_1 + V_2}{2jdb}$ which is equivalent to

the average shear. This vertical shear is at all times equal to the average shear. This vertical shear is also equal to the horizontal shear (which is proven in any standard mechanics book). Thus for a beam of breadth b the total horizontal shear per unit length of beam will be $\frac{V_1 + V_2}{2bdj}$ or considering $Y = \frac{V_1 + V_2}{2}$ there results $\frac{V}{jd}$. For a distance "s" between stirrups each stirrup will be subjected to a stress equal to $\frac{Vs}{jd}$ which is resisted by $a_s f_3$ the strength of the stirrups. Therefore:

$$a_s f_s = \frac{Vs}{jd}$$

The present standard method of design places a coefficient two thirds before $\frac{Vs}{jd}$ as it does not consider that the entire vertical component is taken by the steel, but that the concrete aids it by taking one third of the total stress. This is merely an assumption and the work in hand is being carried out to verify or determine the value of this coefficient.

Outline of Method of Procedure.

Three T beams were built for the purpose of these tests. Two of these contained web reinforcing consisting of vertical stirrups, while the remaining beam had inclined bars to take care of the diagonal tension.

The beams were twelve feet long, placed on twelve inch steel supports; this made the effective length of the beams eleven feet. The width of the flange was limited to twenty-two inches, so that it could be put into the testing machine.

The load was gradually applied, at the third points, until a value of twenty-five thousand pounds was reached. (The beam was designed for this loading.) The twenty-five thousand pounds at the third point was transmitted by knife edges, due to a fifty-thousand pound load applied at the center of two ten inch I beams supported on the knife edges.

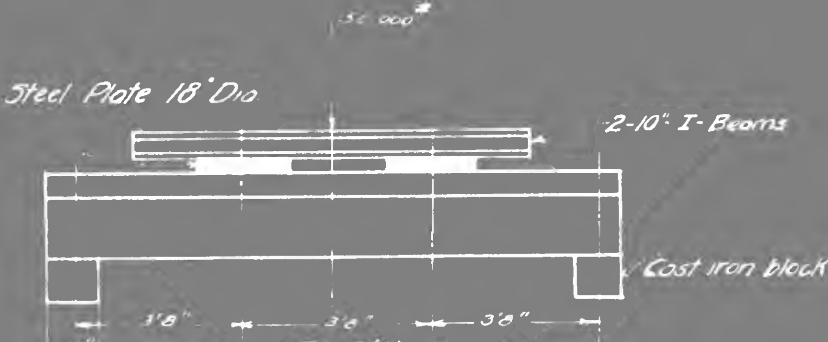


Fig. 8(a)
Method of Loading.

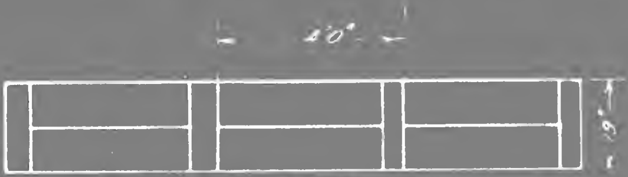


Fig. 10
Side of form without flange.

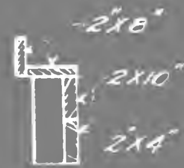


Fig. 11.
Section showing Flange.

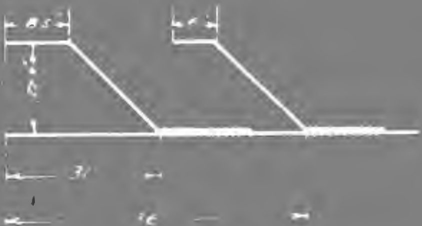


Fig 9(a)
Detail of [illegible] bars

Detail of Stirrup.

Fig. 12

The method is shown in Fig. 8.[a] Most of the recent experimental work upon concrete beams or slabs has been performed by loading these at the third points, because it is claimed that thereby the closest average approximate conditions are encountered. In this arrangement the bending moment remains practically uniform between loads.

The beams were properly reinforced for tension assuming a safe stress of sixteen thousand pounds per square inch in the steel, and a safe compressive stress in the concrete of six hundred fifty pounds per square inch. The stirrups to reinforce the web were designed for a safe stress of thirty thousand pounds per square inch. Hence, since the yield point of the steel is about thirty thousand pounds, failure due to stirrup stresses must occur first. The tensile reinforcement consist of three fourths inch plain round rods

of mild steel; the stirrups were of the same materail but three eighths inch in diameter.

Notation Used.

f_s = safe unit tensile strength of steel.

f_c = safe unit compressive strength of concrete.

Y = total vertical shear.

$n = \frac{E_s}{E_c}$

E_s = modulus of elasticity of steel in tension.

E_c = modulus of elasticity of concrete in compression.

b = width of flange of T beam

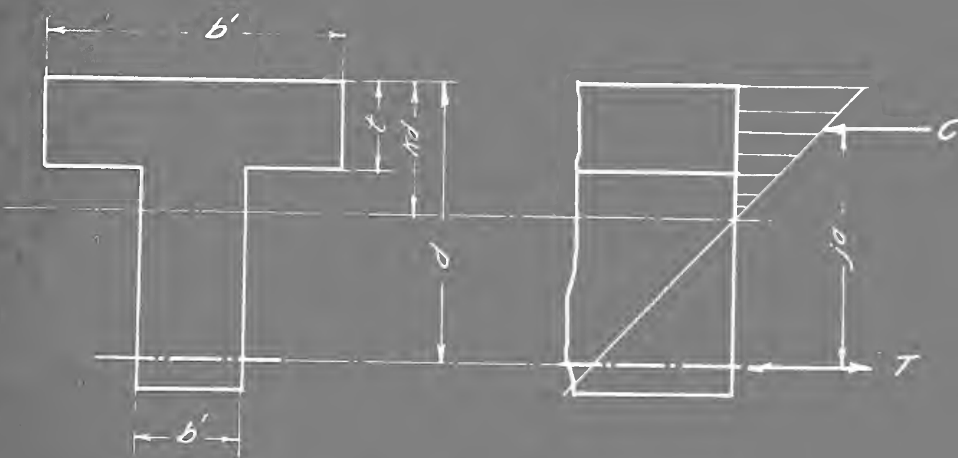
b^1 - width of web.

d = depth of beam from compression face to center of tensile reinforcement.

k = a coefficient which when multiplied by "D" gives the distance from the compression face to the neutral axis.

A_s = area of cross section of steel.

$p = \frac{A_s}{bd}$ = ratio of steel area to the area of the cross section of the beam.



Cross-section showing notation used in design of beam.

M_c = Resisting moment due to concrete.

M_s = resisting moment due to steel.

M = bending moment in general.

j = is the number which when multiplied by "D" given the distance between the center of compression in concrete and the center of tension in steel.

t = thickness of flange.

u = unit bond stress.

o = circumference of one bar.

l^1 = length of embedment of inclined bars at top.

Calculations.

Assume f_s = 16,000 lbs. per square inch.

f_c = 650 lbs. per square inch.

Safe shearing strength in stirrups.

30,000 lbs. per square inch = f_s in stirrups.

n = 15"

b = 22"

d = 23"

v = unit vertical shear = 120 per square inch.

The formulas used are those found in any standard text-book on reinforced concrete design.

$$V = vb^1d \qquad b^1d = \frac{Y}{120}$$

Assume $d = 23"$ Assume total dead load 4000 lbs.

$$b^1 = \frac{27000}{120 \times 23} = 9.8"$$

$$K = \frac{1}{1 + \frac{fs}{nfc}} = \frac{1}{1 + \frac{16000}{15 + 650}} = \frac{1}{1 + 1.64} = .378$$

K_d = $.378 \times 23 = 8.73"$ ∴ case 2 for designing T beams may be used.

$$A_s = \frac{M}{f_s jd} \qquad \text{Assume } \frac{9}{10} \text{ for } j.$$

$$A_s = \frac{11,850,000}{16,000 \times 9 \times 23.75} = 3.58 \text{ sq.inches required.}$$

Using 3/4 round rods which have an area of .4418 square inches. $\frac{3.58}{.4418}$ = 8.1 bars are necessary. Eight bars were used.

The value of "T" was determined from the equation.

$$kd = \frac{2ndas + bt^2}{2na_s + 2bt}$$

$$8.73 = \frac{2 \times 15 \times 23 \times 3.58 + 22t^2}{2 \times 15 \times 3.58 + 2 \times 22 \times t.}$$

$$22t^2 - 2504.7 = 950.69 \pm 384.1t.$$

$$22t^2 - 384.1t \pm 1554.01 = 0$$

$$t = \frac{-17.4 \pm \sqrt{(17.4)^2 - 4\ (70.8)}}{2}$$

$$= \frac{17.4 \pm 4.66}{2} = 11.03'' \text{ or } 6.37'' \qquad t = 6.37'' \text{ was used.}$$

The test for the required bond developed by the rods is determined by the value for

$$u = \frac{V}{eojd} = \frac{27000 \times 10}{8 \times 2.356 \times 9 \times 23} = 69 \text{ lbs.}$$

The allowable bond stress by the Joint Committee in Concrete and Reinforced Concrete is 80 lbs. per square inch.

Design of Stirrups.

Using single loop 3/8" round stirrups the spacing is figured as:

$$S = \frac{3}{2}\frac{A_s f_s jd}{V} = \frac{\frac{3}{2} \times .2204 \times 30{,}000 \times .9 \times 23}{27{,}000}$$

$= 7.6''$ 7.5" spacing between stirrups was used.

The beam actually constructed could not

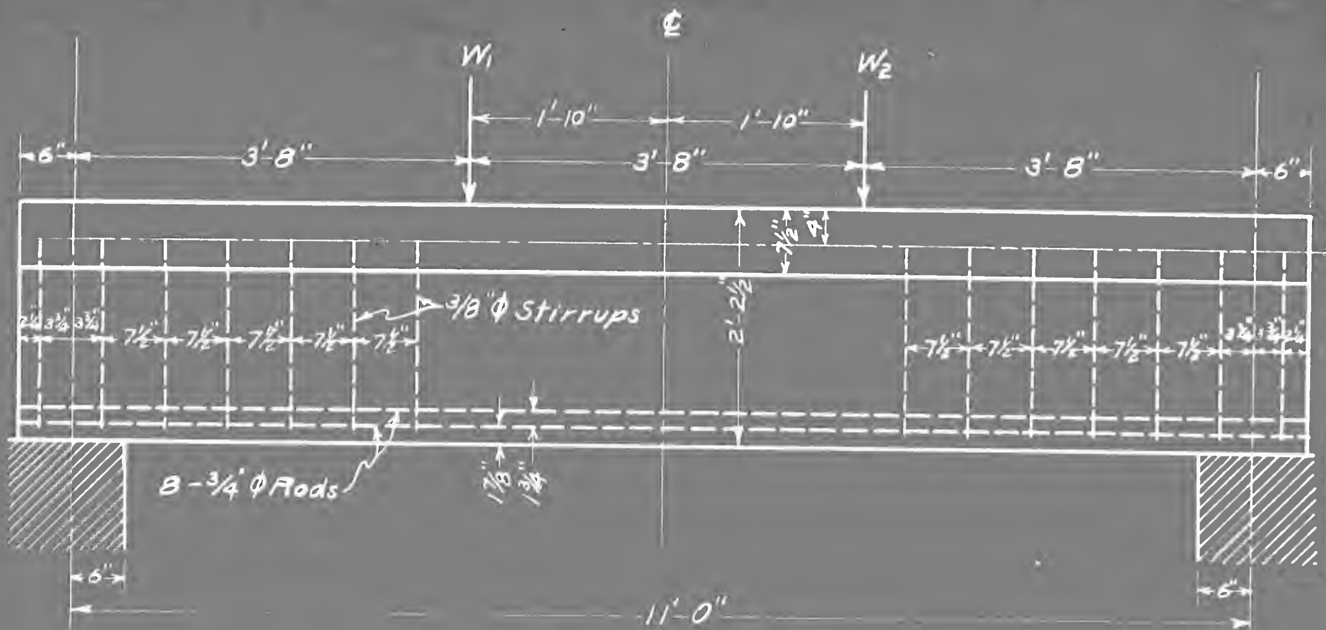


FIG. 12(a)

VERTICAL PLAN OF T-BEAM

be made exactly according to the above calculations as the lumber available for the forms varied first a little from the size necessary to have the above dimensions. The actual dimensions are those shown in Fig. 9.[a]

Making $b^1 = 9.25''$

$b = 21.00''$

$d = 23.75''$

$t = 7.5''$

Stirrup spacing $= 7.5''$

Theoretically stirrups are not necessary beyond the third points as the shear there is nearly zero, except for that due to the weight of the beam. Nevertheless one stirrup was carried beyond the third points, and one beyond each support.

It should be noted that j was assumed as $\frac{9}{10}$. J depends upon values of t, d, p and n. After the determination of n and with the actual value of t, d, p the value of j may be calculated from

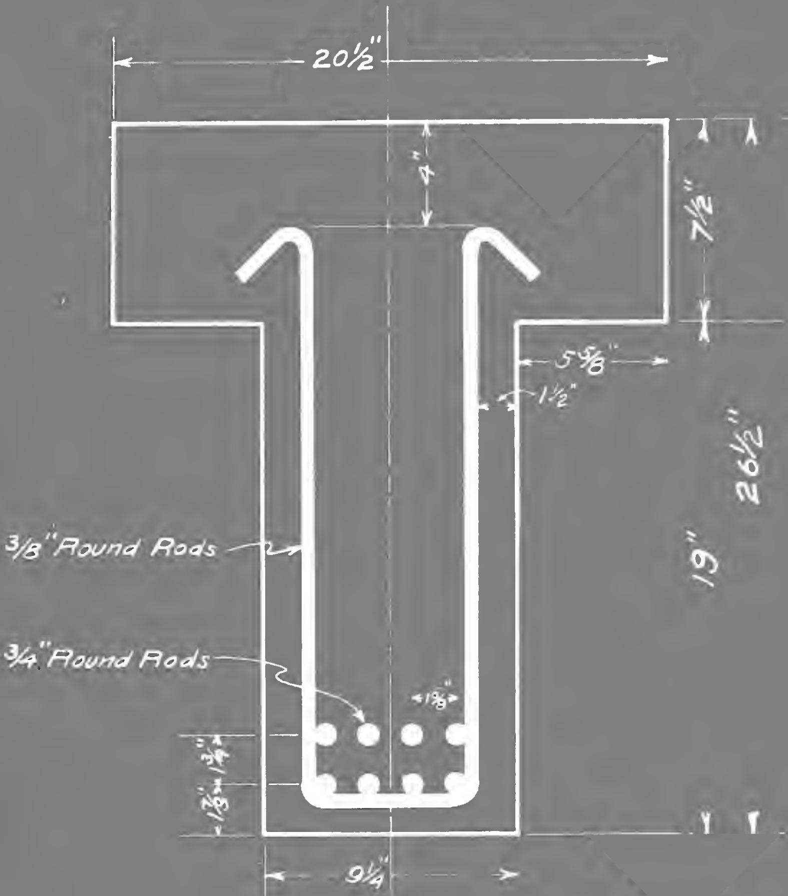


Fig. 9 (d).

Section of T-Beam

$$j = \frac{6 - 6\left(\frac{t}{d}\right) + 2\left(\frac{t}{d}\right)^2 + \left(\frac{t}{d}\right)^3 \left(\frac{1}{2\,pn}\right).}{6 - 3\,\frac{t}{d}}$$

Determination of "N".

The value of n which equals $\frac{E_s}{E_c}$ was determined by testing the samples of steel and concrete, as described later in this account.

Design of Beam with Inclined Rods.

$V = 27{,}000$ lbs.

$l = 11$ ft. 0 inches.

$$\frac{t}{d} = \frac{7.5}{23.75} = .310$$

$$p = \frac{A_s}{bd} = \frac{3.53}{20.5 \times 23.75} = .0071.$$

For $\frac{t}{d} = .310$ and $p = .0071$ from table 9, Hool, Vol. 1. $j = .883$ $k = .368$.

$$\therefore u = \frac{V}{Eojd} = \frac{27{,}000}{2.356 \times 23.75 \times .883} = 482 \text{ lbs.}$$

total bond stress.

Number of bars necessary for bond $\frac{482}{80 \times 1.5} = 4.$

Therefore 4 bars may be turned up.

$$l^1 = \frac{fsd}{4u} = \frac{16{,}000}{120} = 33d.$$

$$= 33 \times \frac{3}{4} = 22.25 \text{ inches.}$$

U in this case should not be allowed more than 840 pounds per square inch.

Horizontal shear $= \dfrac{27000}{.883 \times 23.75} = 1290$ lbs. per lineal inch.

The distance between loads and supports is 46 inches.

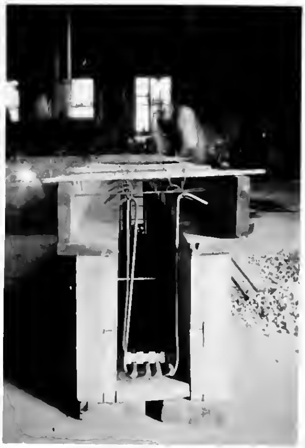
Total horizontal shear is $1290 \times 46 = 59340$ lbs. Assuming that concrete takes one third of the total horizontal shear, $\dfrac{59340}{3} = 19780$ lbs. takmn by the concrete, which leaves 39,560 lbs. stress taken by the web reinforcing.

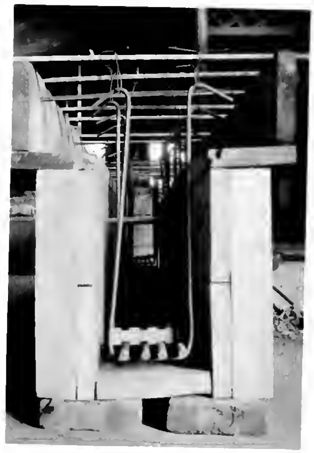
The stress taken by the four inclined rods is $\dfrac{1.76 \times 30{,}000}{.7} = 75{,}500$ lbs.

This beam was constructed by running eight bars through and fastening the inclined reinforcing to the upper layer of tensile bars, spaced as shown in Fig. 9(a).

Construction of Forms.

The forms for the T beams were made of





Forms just before placing of concrete. Note spacing piece used between hori-bars; also method of temporarily sup-porting stirrups.

green pine lumber; the total amount used was 344 board feet, which was ordered as shown:

Number	Nominal Size	Actual Size.
2	2" x 4"	1-5/8" x 3-5/8"
8	2" x 8"	1-5/8" x 9-1/2"
10	2" x 10"	1-5/8" x 7-1/2"

The sides of each form were made of four pieces of 2" x 10" lumber. Each side was constructed of two pieces which were fastened together by means of 2" x 4" struts 19" long placed four feet center to center. Fig. 10 shows this.

For the bottom a 2" x 10" plank was used.

The flange was constructed by nailing together two 2" x 8" boards, at right angles to each other and then nailing this combination to the top of each side so that the right angles faced the inside of the form. Fig. 11 may make this clearer.

Two one quarter inch holes were drilled every three feet in the sides and the flange of

the beam. Through these number eight iron wire was drawn; in this manner the forms were tightened. The holes drilled in the sides of the form were placed two inches and three inches from the projection flange and the bottom of the beam respectively. Those wires placed three inches from the bottom, besides holding the form together served as a support for the tensile reinforcing bars.

The stirrups consisted of 3/8" round rods which were bent into the form of the detail in Fig. 12.

One stirrup was placed 3-3/4" from the center of the support toward the end of the beam. The first stirrup placed from the center of the support toward the load at the third point also was spaced 3-3/4" from the center of the support. The rest of the stirrups between this and the load at the third point were spaced 7-1/2" center to center. Theoretically no stirrups are needed

beyond the load at the third points, but one was placed beyond the load.

The bottom row of tensile bars was then put in place and fastened to the wires in the bottom of the form. The stirrups in turn were fastened at the bottom to the tensile bars. At the top the stirrups were held in place by temporary supports made of lathes, as shown in the photograph. These struts were removed after enough concrete was placed in the forms to hold the stirrups in place.

After the stirrups were supported and properly spaced the second layer of tensile bars were spaced by means of a wooden spacer placed at both ends and the center of the form.

Then the end pieces were put on and the cracks due to the warping of the lumber were caulked, so that a minimum leakage would occur.

MATERIALS.

Sand and Stone.

The sand used was that designated as Torpedo which was supplied by the Garden City Sand Co. It was clean and sharp weighing about one hundred pounds per cubic foot, (loose measurement,) containing about 30% voids.

The stone was of the crusher run type of limestone having about 49% of voids.

Cement

Leheigh Portland Cement weighing ninety-four pounds per cubic foot was used. The test for fineness showed the quality of the sand was such that only 2% was retained on the one hundred mesh sieve, and 16.82% on the two hundred mesh sieve which allowed 82.8% pass into the pan. This agrees with specifications of the American Society of Testing Materials.

Mesh	Retained in	Percent Pasing through
100	1	98%
200	8.4	83.28%

The specific gravity was found to be about 3.1%.

Ultimate Strengths
Cement
Table 1.

Age-Days	Strength Neat Tension	Compression
3	455-373	
7	490-494	18,720
14	532-569	15,720
21	367-499	24,820

Briquetts

1:1	1:3
532-454	258
552-655	142-186
554-533	390-325.

STEEL.

The reinforcing steel consisted of plain round bars of mild steel; 3/8" bars were used for the stirrups, and 3/4" bars were used for the tensile steel. Samples of all the bars were taken and tested, the results being shown in the accompanying table. (Table II)

The modulus of elasticity is found from

the average values of the tests, using the relation $E_s = \frac{pl}{Ae}$ where P is the total stress, A the area of across section, e the elongation, and l the length of the specimen.

To determine these values specimens of the steel were subjected to tensile tests, measurements of loads and elonagtions being noted as shown in the table.

TABLE II

Tensile Test of Reinforcing Steel.

Load Actual P	Per.Sq.In. p	Extension Left
500	7000	.0003
600	8400	.0004
700	9800	.0006
800	11200	.0007
900	12600	.0009
1000	14000	.0011
1100	15400	.0012
1200	16800	.0012
1300	18200	.0013
1400	19700	.0014
1500	21000	.0015
1600	22400	.0016
1700	23800	.0017
1800	25200	.0019
1900	26600	.0021
2000	28000	.0022
2100	29400	.0022
2200	30800	.0022
2300	32200	.0022
2400	33600	.0025
2500	35000	.0027
2600	36400	.0028
2700	37900	.0029
2800	39300	.0030
2900	40600	.0031
3000	42000	.0032
3100	43500	.0033
3200	44750	.0034
3300	46250	.0035
3400	47600	.0036
3500	49000	.0034
4770	67000	Ultimate

TABLE II

Tensile Test of Reinforcing Steel.

Right	Extension Per Inch.	Modulus of Elasticity $E_t = \frac{p}{e}$
.0004	.000170	41250000
.0005	.000225	37400000
.0005	.000275	35500000
.0005	.000300	37400000
.0006	.000375	33600000
.0006	.000420	33000000
.0008	.000500	30800000
.0000	.000525	32000000
.0010	.000575	31700000
.0011	.000625	31400000
.0012	.000675	31200000
.0013	.000750	29800000
.0014	.000775	30600000
.0015	.000900	28000000
.0015	.000925	27200000
.0015	.000925	28800000
.0016	.000950	29500000
.0017	.000985	29900000
.0017	.000985	32800000
.0019	.001100	30500000
.0020	.001175	29900000
.0020	.001200	30300000
.0022	.001275	29700000
.0023	.001325	29600000
.0024	.001375	29500000
.0025	.001425	30000000
.0025	.001450	29800000
.0026	.001500	30400000
.0028	.001525	29800000
.0029	.001600	28800000
.0034	.001700	
Stress		

Original Length	2.000	inches
Final Length	2.390	inches
Total Extension	0.390	inches
Form of Section	Circular	
Original Diameter	0.3015	inches
Original Area of Section	0.0715	sq. in.
Maximum Load	67,000	Lb.per sq.in.
Modulus of Elasticity	28,800,000	Lb.per sq.in.

TABLE II

Tensile Test of Reinforcing Steel.

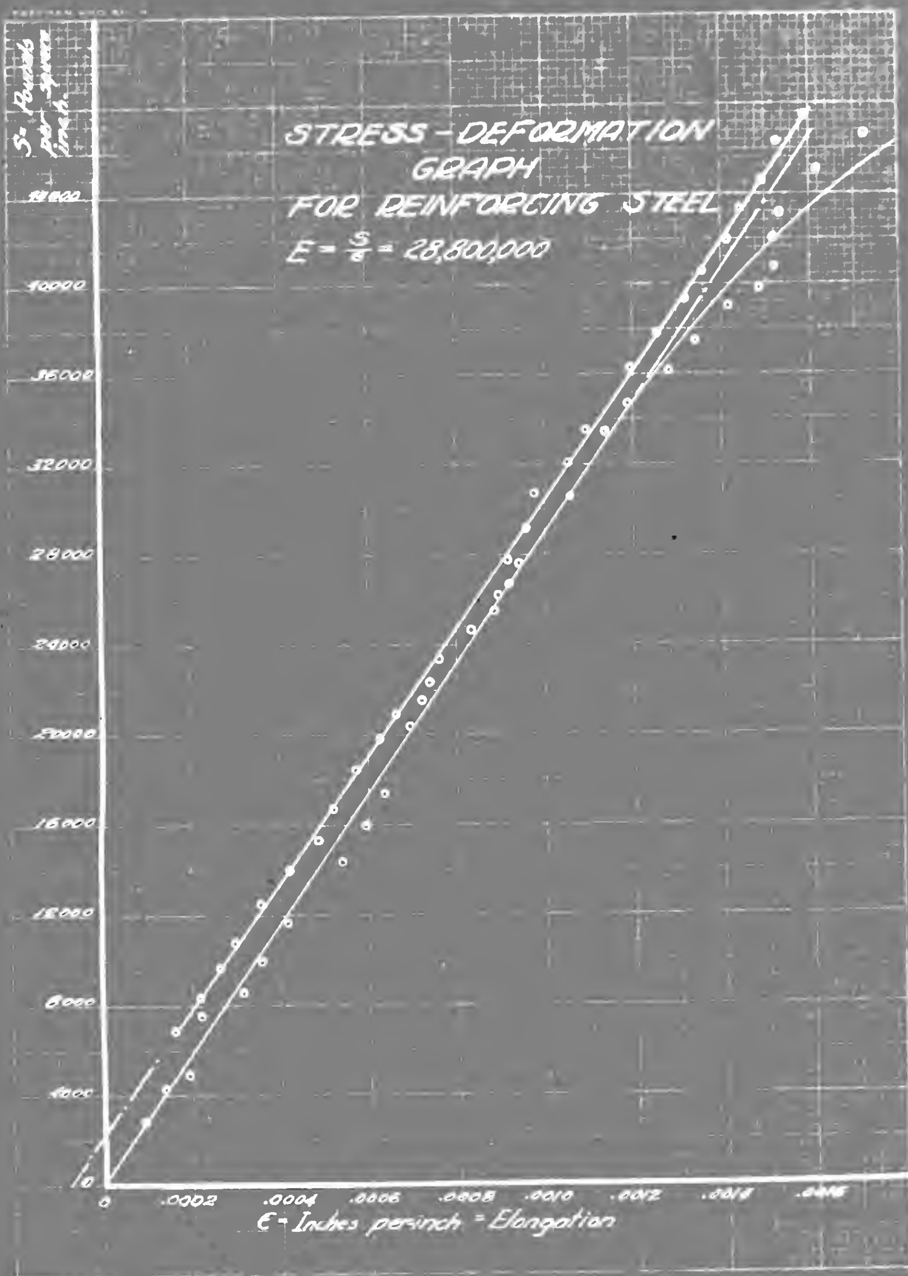
Load Actual P	Load Per.Sq.In. P	Extension Left
200	2900	+.0001
300	4370	-.0001
400	5800	-.0001
500	7270	±.0001
600	8725	.0002
700	10180	.0003
800	11630	.0004
900	13100	.0006
1000	14500	.0007
1100	16000	.0008
1200	17400	.0009
1300	18900	.0009
1400	20400	.0010
1500	12800	.0011
1600	23300	.0011
1700	24800	.0012
1800	26200	.0014
1900	27600	.0015
2000	29200	.0015
2100	30600	.0018
2200	32000	.0018
2300	33400	.0019
2400	34800	.0020
2500	36200	.0021
2600	37700	.0022
2700	39100	.0023
2800	40600	.0024
2900	42000	.0025
3000	43500	.0027
3100	45000	.0030
3200	46500	.0033
3300	48000	

TABLE II

Tensile Test of Reinforcing Steel.

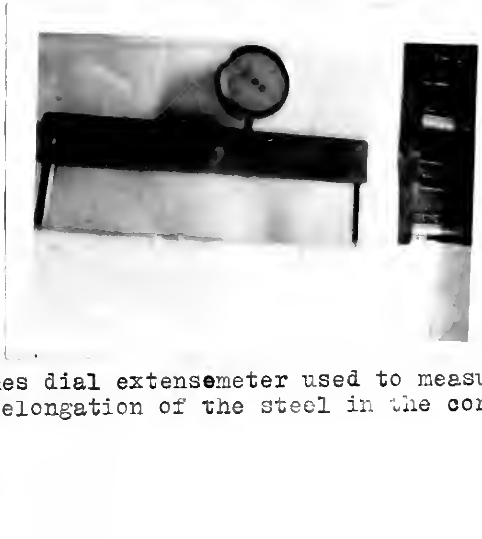
Extension Right	Per Inch.	Modulus of Elasticity $E_t = \frac{p}{e}$
.0003	.00010	29,000,000
.0007	.00015	29,000,000
.0008	.00175	33,200,000
.0010	.000225	32,300,000
.0011	.000325	27,000,000
.0012	.000375	27,200,000
.0013	.000425	27,400,000
.0014	.000500	26,200,000
.0015	.000550	26,400,000
.0016	.000600	26,700,000
.0017	.000650	26,700,000
.0017	.000650	29,200,000
.0018	.000700	29,100,000
.0018	.000725	30,100,000
.0020	.000775	30,100,000
.0022	.000850	29,200,000
.0022	.000900	29,100,000
.0023	.000950	29,100,000
.0024	.000975	29,900,000
.0025	.001075	28,600,000
.0025	.001075	29,800,000
.0027	.001150	29,000,000
.0028	.00120	29,000,000
.0031	.00130	28,000,000
.0032	.001350	28,000,000
.0032	.001325	29,400,000
.0033	.001425	28,600,000
.0034	.001425	29,600,000
.0035	.001550	28,000,000
.0035	.001625	27,600,000
.0036	.001725	27,000,000

Original Length	2.000	inches
Final Length	2.503	inches
Total Extension	0.503	inches
Form of Section	Circular	
Original Diameter	0.296	inches
Original Area of Section	0.06876	sq.in.
Maximum Load	64,000	Lb.per sq.in.
Modulus of Elasticity	28,800,000	Lb.per sq.in.



STRESS - DEFORMATION GRAPH FOR REINFORCING STEEL
$E = \frac{S}{e} = 28,800,000$
S - Pounds per Square inch.
E - Inches per inch = Elongation
0
4000
8000
12000
16000
20000
24000
28000
32000
36000
40000
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016

The first few attempts of measuring the elongation of the specimens were made with a dial type extensometer. The results of testing several pieces of steel by means of this instrument were in no way consistant, very high values being obtained for E. After a little experimentation it was found that this result was due to the unequal elongation of the fibers on both sides of the test piece. The dial type of extensometer is constructed so that the extension on only one side of the specimen may be read. This would be sufficient if the equal elongation would take place on both sides of the sample, but due to any slight eccentricity or any small variation in the uniformity of the material an extensometer which measures the deflection on both sides should be used. Such an instrument was used after the first few trials and the results of several tests showed the modulus of elasticity to be about 28,800,000. This result was



Ames dial extensemeter used to measure
the elongation of the steel in the concrete.

obtained by plotting a stress-deformation curve. A straight line was drawn through the average values of the points plotted and the tangent of the angle that this line makes with the abscissa is the modulus of elasticity. The yield point was found to be about 48,000 pounds per square inch, while the ultimate stress amounted to 65,000 pounds per square inch. The above determinations show that the reinforcing steel is of good quality, and that it may be used with a fair degree of accuracy in serving as an indicator of the stresses in the stirrups.

Casting The Beams

The beams were made in one of the testing laboratories of the Armour Institute of Technology. Two batches of concrete were mixed for the placing of each beam. The volume of a beam was calculated to be about twenty-seven and four-tenths cubic feet;

The first beam as it looked seven days after it was made. The forms were removed then.

hence an amount of stone a little less than this was taken.

It was intended to use a 1-2-4 mixture in all beams but due to an error in calculating the volume of the wheelbarrow the grading of the first beam was a 1-2.34-4.68 mixture. The others were made as proposed. All measurements of sand and stone were made by a wheelbarrow whose volume is 2.93 cubic feet. The cement was measured by the bag, each bag weighing ninety-four pounds and containing one cubic foot.

The proper amount of stone was first measured and then spread out. The proper percentage of sand and c ement were thoroughly mixed, being turned three times. This intimate mixture was then mixed with the stone and turned (dry) twice. Water was added by means of a hose in such amounts that a quaking concrete resulted. Three turns were made of this mixture. This was then placed in the forms

being thoroughly rodded and tamped so that a good bond was formed between the concrete and steel. When a sufficient, amount of concrete was placed so that the stirrups remained fixed in position the upper fastenings to the struts were removed. The temperature for the first few weeks remained about 56° - 65° F.

The forms on the first two beams were removed after seven days and the third after fourteen days.

For the first few days the beams were thoroughly soaked.

Two samples of the concrete from each beam were made up in one foot cubes to be tested on the same day that the beams were tested. The purpose of these tests was to determine the modulus of elasticity of the concrete, so that the proper value of n could be determined. The results of this test is given with account of the testing of the beams.

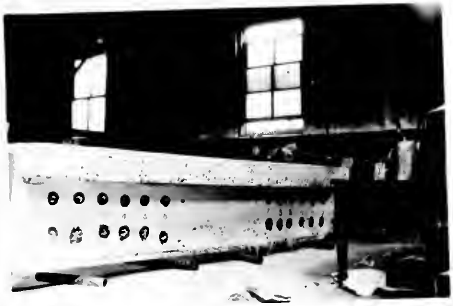
Samples of concrete used inmaking the beams.

Test of Beam 1.

Cast January 28, 1918 Tested May 8, 1918.

Before beam No. 1 could be tested holes had to be drilled into its sides by means of three quarter inch star drills. These holes were placed so as to locate the stirrups and were spaced eight inches apart vertically, the top hole being five inches below the bottom of the flange. By means of these holes an extensometer having an eight inch gauge could be used to measure the deflection of the stirrups. After the holes were made all loose material was removed by means of a pneumatic blower. The exposed stirrups were then washed so that any markings on the steel could be readily distinguished.

Then the whole beam was given a coat of whitewash so that any slight cracks could be easily seen. Black circles were painted around the openings and each set of openings was numbered. As only twelve stirrups were



Moving the beams by means of rollers.

Cast iron plate used as a turn table.

exposed the openings were numbered from one to twelve on both sides.

By means of two inch pipe rollers, crowbars, and a large jack the beam was rolled underneath the four hundred thousand pound Riehle testing machine. It was placed so that its sides faced north and south and any stirrups hereafter mentioned will be designated as N_1, N_2, S_1, S_2, etc. The beam was supported upon cast-iron blocks ten inches wide, twelve inches long, and twelve inches high. The loads at the third points (three feet eight inches from the center line of the supports) were applied by two circular steel plates about eighteen inches in diameter and two inches in thickness. These plates received the load from two ten inch I beams which were loaded at the center and supported on the plates.

Before applying the load punch marks were made upon the stirrups with a two punch

points eight inches apart. Small circles were put around these marks with a red pencil so that the indentations could be readily found later. Then the zero readings of the extensometer were taken and recorded.

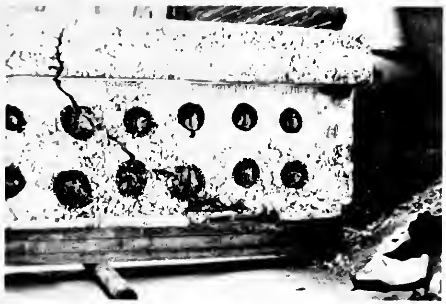
The load was applied in gradual increments of five thousand pounds. When the desired value of the load was reached it was kept at that value until the extension of all the stirrups was recorded. The extension was measured by means of an Aimes dial placed upon an eight inch gauge extensometer. This instrument was placed in the markings made upon the steel, and hence measured any elongation present. It took about thirty five minutes to take one complete set of of readings, hence the load applied was concentrated for that length of time. No definite elongations were noted at any of the loads up to sixty-five thousand pounds. Slight variations from .0001 to .0002 were

Top view: Measuring elongations in inclined bars.
Bottom view: Measuring the elongations in the stirrups.

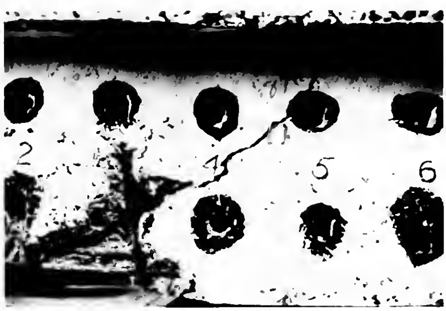
noted, but these deformations were not in the same direction being tension at some loads and compression at others. Since these quantities are so small it is doubtful whether these changes were really due to the stress in the stirrups or due to variations in the readings, which may occur in this method of measuring the extension. Repeated trials, however, showed that the error due to reading may be neglected.

At sixty-five thousand pounds a fine crack starting at the bottom and extending as far up as the tensile reinforcement was found directly under stirrup N_3. The same type of crack was found at a point eight inches from the load point toward the center of the beam.

Another fine crack was found beginning at the bottom about two inches to the right of N_8 and extending diagonally across the face of the beam to the flange about two inches to



North side of beam No.1, where failure occurred at a load of 87, 000 pounds.



South side of beam showing same failure. Concrete was removed to show that stirrup did not fail.

the left of the same stirrup. No other signs of giving way were noted at this load.

At seventy-five thousand pounds failing in the web became more distinct being shown in Fig. 14. The cracks began at about two inches from the bottom and extended only to approximately the center of the depth. These occurred on the south side of the beam; on the north side a small diagonal crack formed at N_4.

This beam was designed for a safe bending stress due to twenty-five thousand pounds applied at the third points, but the web of the beam was designed so that failure would occur in it at this loading.

Up to this loading no elongations in the stirrups were noted; not even in those stirrups near which the diagonal tension failure was taking place. According to the formula $A_s f_s = \frac{2}{3} \frac{V_s}{Jd}$ the stress in stirrup No. 4 would be thirty-nine thousand eight

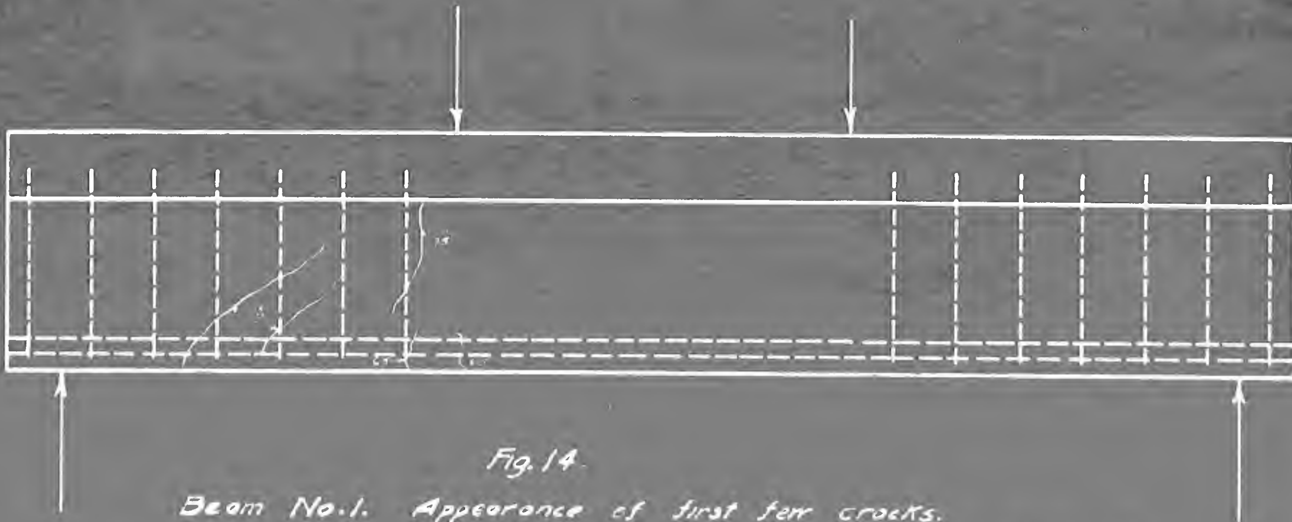
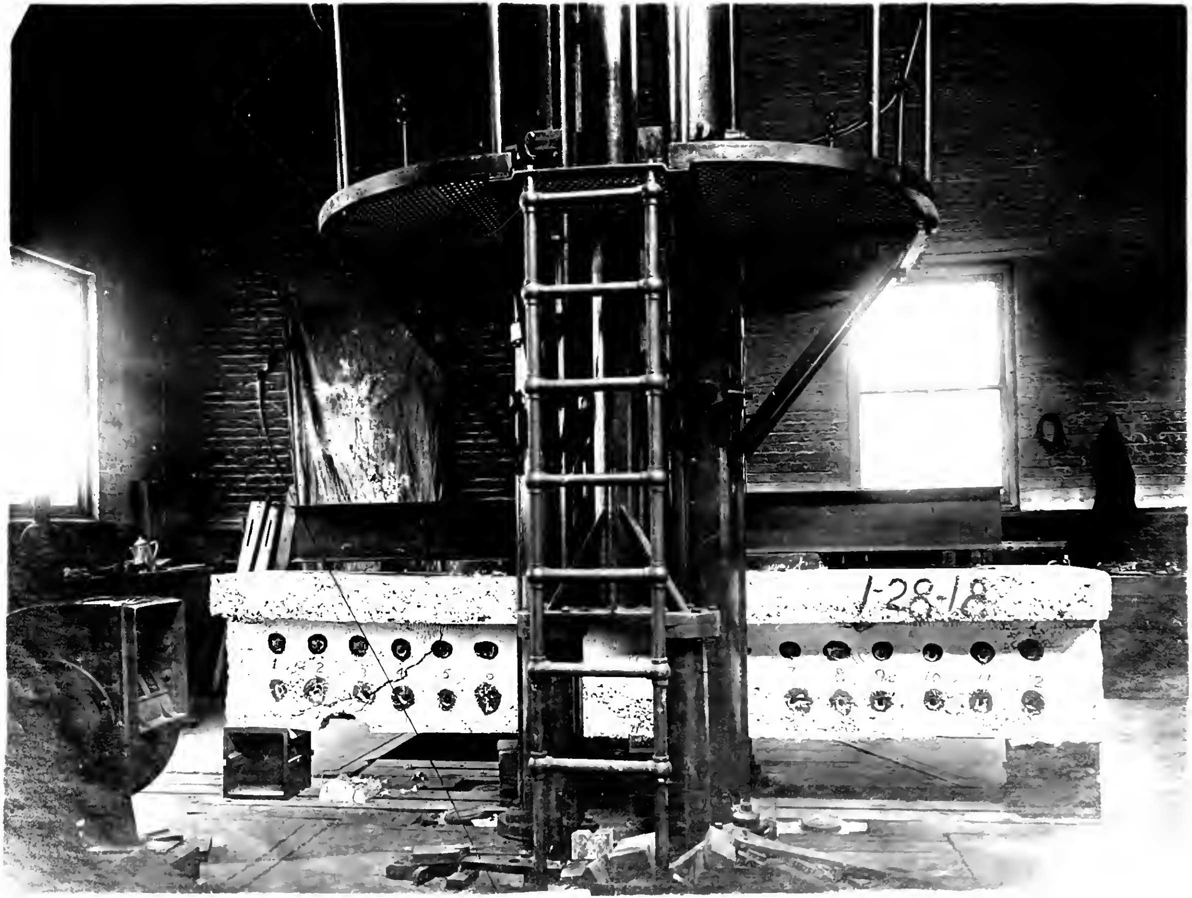


Fig. 14.

Beam No. 1. Appearance of first few cracks.
Numbers indicate load in thousands of pounds.

hundred pounds per square inch. With a tensile stress of this magnitude and a modulus of elasticity of twenty-eight million eight hundred thousand an elongation of .001385 inches should take place and this would indicate a reading of 6.92 divisions in the Aimes dial. But no such results were obtained at this load which shows that although the concrete is receiving a good deal of stress the stirrups are subjected to no direct tensile stress as theoretically calculated.

At eighty-seven thousand pounds a distinct diagonal tension crack began to form beginning at about twelve inches from the west support, extending about two inches vertically, and then diagonally across the face of the beam up to the loaded point. This crack occurred on both sides of the beam and across the flange. It was three sixteenths of an inch wide at the top and



Beam No. 1 after failure at a load of [illegible] pounds.

and about three quarters of an inch wide at the bottom. The concrete failed as the arm of the testing machine dropped and the latter could not be made to rise without reducing the load. The arm rose for a short time at fifty-four thousand pounds and kept dropping until the load was reduced to forty-eight thousand pounds where it remained somewhat constant. The elongations in the stirrup N_4, S_4, became prominent now, as the concrete had failed and the steel was taking the total stress.

The tensile stress in the stirrup now may be attributed to the load which the tensile rods apply at the bottom of the stirrup due to the stress in the horizontal rods. Since the stirrups are well hooked at the top, this action induces the tension noted in the stirrup. Thus before the stress in the horizontal rods became excessive no stresses were noted in the stirrups.

When the load at the center of the I beams was eighty-seven thousand, the accompanying table shows that there was some deflection on both sides of stirrups 2-3-4-5, all of which are in the direct path of the diagonal tension crack. The actual stresses in these stirrups as calculated from the formula $S = eE$ and the theoretical stress for No. 4 are shown in the table.

It can be seen that the stirrups do not take two thirds of the total stress and that the concrete takes one third; the concrete takes a much high stress, but its relations to the total stress cannot very well be determined from this data, as the figures show no actual stress in the steel until the concrete has entirely failed.

For stirrup No. 4 which shows the maximum elongation the following results are shown: On the north side the total deflection was .00072 inches and on the

south side .0086 inches (for a load of eighty-seven thousand pounds). The average deflection per unit length was .000987 inches per inch. The stress calculated from S = E_e was 28,400 pounds per square inch. The stress calculated from $f_s = \frac{Y_s}{Jd \times A_s}$ equals (Assuming that f_x carries the total stress) sixty-nine thousand three hundred pounds per square inch. This gives a ratio of twenty-eight thousand four hundred to sixty-nine thousand three hundred as 1: 2.44. This shows that the steel took only 0.41 of the total stress and that the concrete took the rest. These calculations do not include the dead laod, but those in Table 111 (a) do include the dead load of four thousand pounds.

TABLE III

128 Day Test of Beam No. 1.
Extensometer Readings of Elongation of Stirrups.

To obtain deflection at any load, subtract reading at that load from zero reading and multiply by .0002.

South Side of Beam.

Stirrups Located by Number.

Load	1	2	3	4	5	6
o	66.4	59.0	61.0	44.5	52.2	45.0
5000	67.2	60.0	60.5	43.5	53.0	44.5
10000	66.0	60.5	61.0	44.5	52.5	45.5
15000	66.5	59.5	61.0	44.5	53.5	46.0
20000	66.5	59.5	61.0	43.5	54.0	46.0
25000	67.5	61.0	61.5	45.0	55.0	47.0
30000	67.0	60.0	61.5	45.5	54.5	48.0
35000	66.5	60.0	60.5	45.5	55.0	47.5
41500	66.0	60.0	59.5	44.5	53.5	46.0
45000	66.5	60.5	60.5	43.0	54.5	47.0
50000	66.4	59.5	59.5	45.0	53.5	47.0
55000	68.0	60.0	60.0	45.0	54.0	47.5
60000	68.0	60.0	59.5	45.5	52.5	47.0
70000	68.0	60.0	56.5	42.5	51.0	47.0
75000	68.0	60.0	55.5	42.0	50.0	47.0
80000	68.0	60.0	54.0	41.5	50.0	47.5
85000				37.0	49.5	
87000	68.5	56.0	42.0	1.5	49.0	46.0

TABLE III

128 Day Test of Beam No. 1.
Extensometer Readings of Elongation of Stirrups.

To obtain deflection at any load, subtract reading at that load from zero reading and multiply by .0002.

South Side of Beam.

Stirrups Located by Number.

7	8	9	10	11	12
72.00	75.00	52.00	62.50	41.25	61.00
73.00	77.00	53.00	62.50	43.00	60.50
73.00	76.00	52.50	62.50	43.00	62.00
72.00	75.00	51.50	62.00	41.50	61.00
72.00	75.00	52.00	62.00	41.00	62.50
73.80	76.50	52.00	63.00	43.00	62.50
74.50	77.00	53.00	63.00	43.50	63.00
73.00	78.00	53.00	63.50	43.50	63.00
72.00	76.50	52.00	62.50	43.00	63.50
74.00	76.50	52.00	62.50	42.50	63.50
74.00	77.00	52.50	63.00	43.50	63.00
74.00	76.50	53.00	63.00	43.50	64.00
74.00	76.50	53.00	63.00	43.50	63.50
74.50	77.00	53.00	63.00	44.00	63.50
73.00	76.00	52.00	63.00	44.00	63.00
73.00	76.00	53.00	63.00	44.50	64.00
72.00	76.00	46.50	63.00	44.50	64.00

By load is meant the load applied at center of I Beams, thus only of one half of it is applied at the third points.

North Side of Beam.

Load	1	2	3	4	5	6
0	79.5	59.70	79.00	57.50	54.00	
5000	79.0	60.00	79.00	58.50	54.50	
10000	80.0	59.50	77.50	58.50	54.50	
15000	80.5	60.00	79.00	58.50	54.50	
20000	81.0	59.50	79.50	61.60	55.00	
25000	81.0	61.00	80.00	60.50	56.00	
30000	81.5	60.50	81.50	59.50	55.50	
35000	81.0	60.50	80.50	59.50	55.00	
41500	79.5	60.50	79.00	58.00	53.00	
45000	81.0	60.00	80.00	60.00	55.00	
50000	81.0	60.00	79.50	58.00	53.00	
55000	81.5	61.00	80.00	59.50	53.00	
65000	81.5	61.50	78.00	59.00	52.00	
75000	82.0	62.00	77.00	56.00	49.50	
80000	82.0	62.00	76.00	55.50	49.00	
87000	82.0	58.50	52.00	21.50	48.00	

7	8	9	10	11	12
86.00	68.00	59.50	76.20	65.00	58.00
87.50	71.50	61.50	77.50	67.00	60.50
89.00	70.00	60.50	74.80	65.00	59.00
89.00	70.00	61.00	76.00	67.00	60.00
89.50	72.00	62.00	78.00	67.50	61.00
90.00	72.50	61.50	78.50	67.50	62.00
89.50	72.00	60.00	75.80	66.50	59.50
89.50	72.00	60.50	77.00	66.50	60.00
89.00	71.50	61.50	77.00	66.50	60.50
90.00	72.50	61.50	77.00	67.00	61.00
89.50	72.50	61.50	78.00	67.50	61.00
90.00	73.50	62.00	78.00	67.00	62.00
90.00	72.50	62.50	77.50	68.00	61.50
88.50	73.00	62.00	78.00	68.00	62.00
88.00	71.00	61.50	76.50	68.00	61.50

Note: No measurements taken on No. 6 side of beam.

TABLE III (a)

Calculations for Stirrup No. 4.

Load	Total Shear	Unit Av.Sh.	Av.Unit Elong.	[1]Actual Stress
0	000	6.1	0	Small
60,000	32000	97.6	0	"
70,000	37000	112.5	.00005	1440
75,000	39500	120.0	.00005	1440
80,000	42000	127.5	.000063	1800
87,000	45500	138.0	.000987	28,440

[2]Theor. Stress	Ratio 1:2
31.9	
51,000	
59,000	.0244
63,000	.0228
67,000	.0238
72,500	.3920

Test of Beam No. 2.

Cast February 23, 1918   Tested May 15, 16, 1918.

Age of Beam   82 Days.

Beam No. 2 was designed and built as beam No. 1, except that the concrete was an exact 1-2-4 mixture.

The preparations for this test were made as described in the test of beam No. 1. Due to the warping of the bottom of the form an uneven bearing surface had developed at the supports. To eliminate this the beam was set upon the supports in a bed of plaster of Paris. The irregularities thus were taken care of.

Before the beam was placed under the machine the scale of the testing machine was adjusted so that the scale arm was balanced at a zero reading. After the T beam had been set in place and the necessary I beams and plates put upon it, the scale reading was found to be five thousand five

hundred pounds. The dead weight of the concrete and I beams, however, was only five thousand two hundred pounds, as three hundred pounds had to be deducted for the weight of the two C. I. block supports.

Initial readings were taken and the load was then applied gradually until the scale reading became twenty thousand pounds. A set of readings was taken at this load. Readings were taken for increments of load of ten thousand pounds until seventy thousand pounds was reached; the next data was taken with an increase of only five thousand pounds each time until the scale indicated a total of eighty thousand pounds. From this value of loading to a value of one hundred four thousand pounds, increments of three thousand pounds were taken. No rapid changes being noticeable in the steel, again intervals of five thousand pounds were used until one hundred fifty four thousand

pounds was reached. A value of one hundred fifty thousand pounds was obtained when the power was shut off, and the test had to be suspended for the day. The load was allowed to remain upon the beam over night for a period of twenty hours.

The next morning the arm of the testing machine was found to have dropped, and a balance was obtained at one hundred twenty seven thousand pounds. Before any more load was applied, a set of readings was taken to determine whether there had been any noticeable effect upon the steel. In the places where the concrete had shown a tendency to fail some elongation was found. Stirrup No. 9 showed a total elongation of .0014 inches.

The load was then slowly increased so that the machine could be balanced. Increments of five thousand pounds were taken until one hundred fifty seven thousand was again reached. This time the machine could

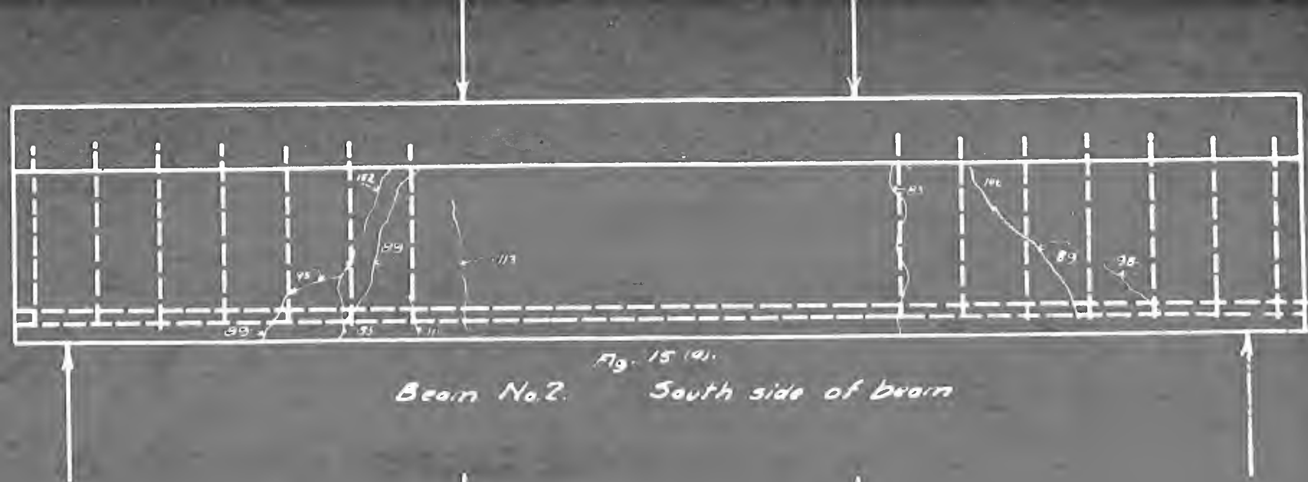


Fig. 15 (a).

Beam No. 2. South side of beam

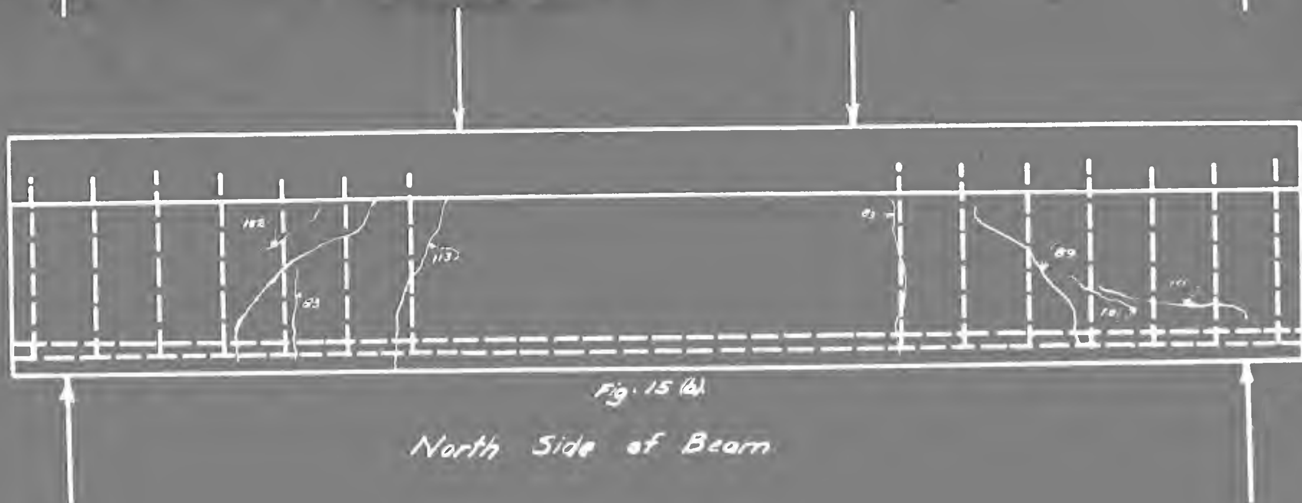


Fig. 15 (b).

North Side of Beam

not be balanced at this load; the load now began to drop away; a temporary balance was obtained at one hundred thirty three thousand pounds. Readings were taken at one hundred thirty three thousand pounds, but before the whole set was obtained the load had dropped to one hundred twenty two thousand pounds. Even now complete failure did not occur as a final balance was obtained at seventy two thousand pounds. At this load the test was concluded.

The first signs of failure occurred at a load of eighty three thousand pounds. Very small, almost unnoticeable cracks formed as follows: One was found at about one half inch west of S_8, extending nearly vertically from the bottom for about five inches. A similar fine crack was noticed about one half inch east of N_8, running vertically to about the middle of the beam. Another such set of failings was

Beam No. 2. Load, 157,000 pounds. Typical diagonal tension crack on South side of the beam.

noted under S_6, N_6; fine cracks beginning at the bottom of the web running within five inches of the bottom of the flange; these were not the beginning of diagonal tension failures.

At eighty-nine thousand pounds the first signs of diagonal tension appeared in two fine cracks on both sides of the beam. On the north side of the beam a fine crack beginning about two inches west of N_3 extended diagonally to a point about four inches below the flange and about two inches east of N_5. This crossed N_4 at eleven inches from the bottom of flange. On the south side of the beam a corresponding opening appeared running from a point one and one half inches west of S_3, two inches from the bottom to a point two inches east of S_5, and four inches from the bottom of the flange. Another crack branched from the first one found under S_8 and N_8; this

began directly under N_8 and S_8, extending diagonally to a point about thirteen inches below the flange and about two inches west of stirrup No. 7. The manner in which other cracks appeared can be readily seen from Fig. 15 (a) and (b). All of these cracks were fine, and none of them extended through at the bottom. When one hundred fifty-seven thousand pounds had been reached the crack shown in the photograph (running from the bottom toward the load point) was not opened much, and the beam showed no definite signs of failure.

The readings taken were very reliable as different methods of checking them always brought the same results.

The extensometer readings showed no marked increased extension in the stirrups until a value of eighty-nine thousand pounds was reached; it will be noted that this is the value at which the first diagonal cracks

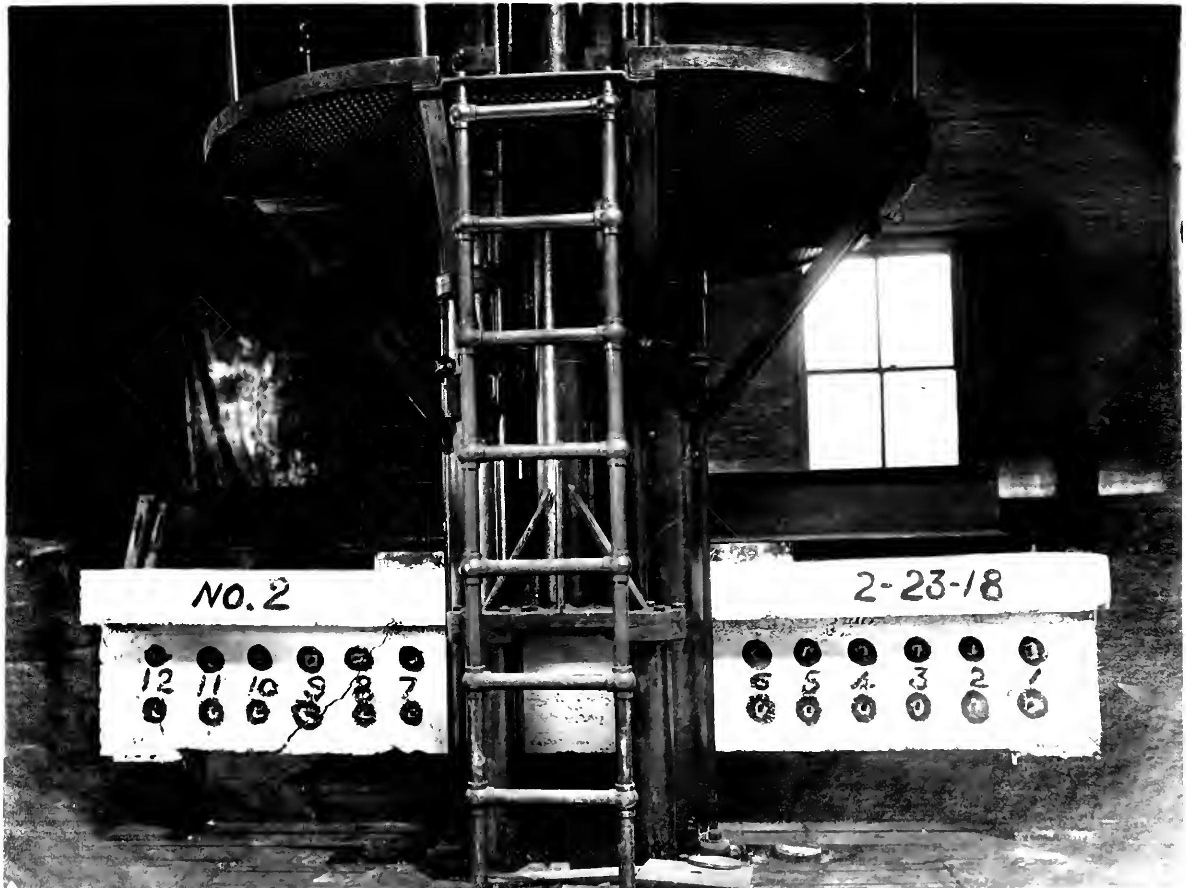
became visible and the steel at these points took a little stress. Up to this load, however, no tension in the steel was noted, although a slight compression took place in most of the stirrups; this was quite pronounced in the stirrups nearest the supports. After a slight failure began in the concrete, this compression in the stirrup gave way, and tension took place, in all stirrups excepting those at the supports. These still remained in compression. The total elongation or compression of nearly all these stirrups was very small except those which are crossed by the diagonal tension crack. The stirrups nearest to the loading points show least change.

An interesting feature took place after the concrete had failed at one hundred fifty-seven thousand pounds. As is seen from the picture failure took place in the diagonal crack between stirrups No. 8 and No. 9. Both

of these stirrups showed elongation at this load but stirrups No. 4 and No. 5 on the other side of the beam show shortening. This is due to the fact that when the failure occurred most of the stress was thrown into the stirrups on the side where the failure of concrete took place, and somewhat releasing the stress on the other side. Since the steel had not passed the elastic limit in stirrups No. 4 and No. 5 there was a tendency for it to return to its original position.

Another note should be made here, that although stirrup No. 4 showed an elongation, no crack appeared across the surface of the beam at that point.

It is seen that the stirrups were stressed more after the load had been released and then again increased to one hundred fifty-seven thousand pounds, than in the first run to one hundred fifty-seven thousand pounds.



NO. 2
12 11 10 9 8 7
2-23-18
6 5 4 3 2 1

The reason for this is that the concrete had become weakened, and more stress had been thrown into the steel.

Calculations of the maximum tension taking place shows that only a small part of the stress calculated is actually developed by the stirrups.

For example, the maximum elongation occurred in stirrup No. 9 at one hundred fifty-seven thousand pounds (repeated load). The unit elongation at this point was .00177 inches; the actual stress was fifty thousand nine hundred seventy-six pounds per square inch, which shows that the steel was stretched beyond the elastic limit. The stress which should occur to develop the total shear according to the theoretical formula is one hundred twenty-six thousand pounds per square inch. This shows that the steel took less than one half of the total stress, and this, after the load had been

repeated. The first time one hundred fifty-seven thousand pounds had been reached the actual stress developed in the stirrup was only seventeen thousand six hundred pounds per square inch.

Calculations before repetition of load.

Stirrup No. 9.

	North Side.	South Side
Zero Reading	83.5	86.6
157,000	58.0	63.5
Elongation	25.5	23.5

Average = $\frac{25.5 \text{ plus } 23.5}{2}$ = 24.5 divisions.

Unit elongation = $\frac{24.5 \times .0002}{8}$ = .000613 in. per inch.

Stress = .000613 x 28,800,000 = 17,600 lbs. per square inch.

Calculated stress by formula.

$A_s f_s = \frac{V_s}{jd}$ $\quad f_s = \frac{7.5 \times 78,500}{.22 \times .9 \times 23.75}$ = 126,000 lbs. per square inch.

Stress: f_s = 17,600 : 126,000 = .1395.

<u>Calculations after load had dropped back to one hundred twenty-seven thousand pounds, and had been brought again to one hundred fifty-seven thousand pounds.</u>

	North Side	South Side.
Zero Reading	83.5	86.0
157,000	0.0	17.5
Elongation	83.5	68.5
Average Elong.	71 Divisions	
Unit Elongation	$\frac{71 \times .0002}{8}$ =	.00177 inches per inch.

Stress = .00177 x 28,800,000 = 50,976 lbd. per square inch.

$$\frac{50,976}{126,000} = .403.$$

The steel in this case is stretched beyond the elastic limit. The average unit shear at this load is two hundred thirty-eight pounds per square inch. Thus it becomes quite obvious that the concrete develops a great deal more than one third of the total stress due to the vertical shear in the beam.

TABLE IV

82 Day Test of Beam No. 2.
Extensometer Readings.

To determine total elongation at any load, subtract reading at that load from first reading and multiply by .0002.

South Side of Beam

Numbers Indicate Position of Stirrups.

Load	1	2	3	4	5	6
5,500	89.0	5.5	94.8	92.2	90.0	10.0
20,000	91.0	5.5	93.5	92.5	90.0	9.5
30,000	90.0	5.0	94.0	93.0	90.5	9.5
40,000	90.0	5.0	94.0	93.0	90.5	10.0
50,000	90.0	5.0	94.5	92.5	90.5	10.0
60,000	90.0	6.0	94.0	93.5	91.5	9.5
75,000	90.5	6.0	94.0	93.0	90.0	10.0
83,000	91.5	6.0	93.5	92.5	91.0	9.5
89,000	90.5	6.0	94.0	85.2	90.5	9.0
92,000	90.0	5.0	94.0	84.5	90.0	9.0
95,000	90.0	5.5	94.5	84.0	89.5	9.0
98,000	90.0	5.0	94.0	83.0	90.0	10.0
101,000	90.3	6.0	93.2	84.0	90.5	9.5
109,000	90.0	5.5	94.0	82.0	90.0	9.5
119,000	91.0	5.5	94.0	80.0	89.5	9.5
124,000	91.0	5.5	94.0	80.0	89.5	9.5
129,000	91.0	5.5	92.5	79.0	89.5	9.5
134,000	91.0	5.5	92.5	78.0	87.0	9.5
139,000	91.0	5.5	92.5	77.5	86.0	9.5
144,000	91.0	5.5	92.5	77.5	86.0	9.5
154,000	91.0	5.5	92.0	76.5	86.0	9.5
157,000	92.0	3.5	92.0	76.0	84.5	9.0
142,000	92.0	3.5	92.0	76.0	84.0	9.0
147,000	92.0	3.5	92.0	76.0	84.0	9.0

Load	1	2	3	4	5	6
152,000	92.0	3.5	92.0	74.5	84.5	9.0
157,000	91.5	3.5	92.0	78.2	84.5	9.5
133,000						

7	8	9	10	11	12
76.5	92.5	86.0	90.0	7.5	97.5
76.5	94.0	86.5	89.5	7.5	97.0
76.0	94.0	86.0	89.5	6.5	97.0
76.2	94.0	86.0	89.5	7.0	97.0
76.5	94.0	86.0	90.0	8.0	97.0
76.5	94.5	85.5	90.0	8.0	98.0
76.5	93.0	86.5	89.0	8.0	97.8
75.5	94.0	86.5	89.0	8.0	97.5
76.0	92.0	84.0	89.0	8.0	98.0
76.0	91.0	84.0	89.0	8.0	97.5
76.0	91.0	83.5	90.0	8.0	97.0
76.0	91.0	83.0	90.0	8.0	98.0
76.0	90.5	84.0	90.0	8.5	98.5
76.0	90.5	82.5	90.0	9.0	98.5
76.0	89.0	79.5	89.0	8.0	98.0
76.0	86.5	79.0	89.0	8.0	98.0
76.0	86.5	77.5	89.0	8.0	98.0
76.0	84.0	75.5	88.5	8.0	98.0
76.0	84.0	74.0	89.0	8.0	98.0
76.0	83.5	73.0	89.0	8.0	98.0
76.0	83.0	72.0	89.0	8.0	98.0
75.5	81.0	63.5	87.0	8.0	98.5
75.5	79.0	61.0	89.0	8.9	98.5
75.5	80.0	59.0	87.5	8.0	98.5
76.0	78.5	52.0	88.0	8.0	98.5
75.5	71.5	17.5	87.5	8.5	98.5

North Side of Beam.

Load	1	2	3	4	5	6
5,500	98.5	91.5	87.5	22.0	81.5	80.0
20,000	97.2	92.0	88.5	22.0	81.5	81.0
30,000	97.5	91.5	87.5	21.0	81.5	80.5
40,000	97.1	91.0	87.2	19.0	81.5	81.0
50,000	97.5	92.0	87.0	19.0	81.5	81.5
70,000	98.5	91.5	87.0	19.0	81.5	81.0
80,000	98.0	91.0	88.0	19.0	81.5	81.0
86,000	98.0	91.5	87.0	18.5	82.5	81.0
89,000	98.5	91.5	86.0	11.5	80.0	80.0
92,000	98.0	91.5	85.5	9.5	81.0	79.5
95,000	98.5	91.5	85.5	9.0	81.0	79.5
98,000	99.0	92.0	85.0	8.5	81.0	79.5
104,000	99.0	91.5	85.0	9.0	82.5	80.0
114,000	98.5	90.5	83.5	8.0	81.0	80.0
119,000	98.5	90.0	83.5	6.5	81.0	80.0
124,000	98.5	90.0	83.5	6.5	81.0	80.0
129,000	98.5	90.0	83.5	6.0	81.0	80.0
134,000	98.5	90.0	81.0	5.0	80.0	79.0
139,000	98.5	90.0	81.0	5.0	80.0	79.0
144,000	98.5	90.0	81.0	5.0	80.0	79.0
154,000	98.5	90.0	80.0	2.0	79.5	79.0
157,000	98.5	89.5	79.5	0.0	78.5	79.0
142,000	98.5	89.5	79.5	0.0	78.5	79.0
147,000	98.5	89.5	79.5	0.0	78.5	79.5
152,000	99.0	89.5	80.0	0.0	78.5	79.5
157,000	99.0	90.0	80.0	2.5	78.0	79.5

North Side of Beam.

7	8	9	10	11	12
94.0	75.5	83.5	9.5	97.0	0.0
94.5	77.2	83.5	9.0	96.5	1.0
93.5	76.0	83.5	9.0	97.0	2.0
94.0	76.5	83.5	9.0	97.5	2.0
94.5	77.0	83.5	8.5	97.5	2.5
93.5	77.0	83.5	9.5	97.5	2.5
93.5	76.5	83.5	9.5	97.5	2.0
93.0	76.0	82.0	9.5	97.5	3.5
93.0	74.3	79.2	9.5	97.5	4.0
92.5	73.0	78.5	9.5	97.5	2.5
92.5	73.5	78.0	9.5	97.5	4.0
92.5	73.5	78.0	9.5	97.5	4.0
92.5	73.0	77.0	9.5	97.5	3.5
92.5	72.3	76.0	10.0	97.5	4.0
92.5	70.5	73.5	9.5	97.5	5.5
92.5	70.5	73.5	9.5	97.0	5.5
92.5	69.0	71.8	8.5	97.5	5.5
92.5	67.5	71.0	8.0	97.5	5.5
92.0	67.0	70.0	8.0	97.5	5.5
92.0	67.5	68.5	8.0	97.5	5.5
92.0	67.5	67.0	8.5	97.5	6.0
91.0	65.0	61.0	9.0	97.5	6.0
91.0	64.5	58.0	9.5	97.5	6.0
92.5	63.5	57.0	9.5	97.5	6.0
92.5	63.5	52.5	9.5	97.5	6.0
92.5	60.5	00.0	9.5	97.5	6.5

Test of Beam No. 3.

Cast March 9, 1918 Tested May 20, 1918.

Age of Beam 72 Days.

Web Reinforcement Inclined Bars.

Beam No. 3 was constructed differently from the first two inasmuch as it contained web reinforcement of inclined bars instead of vertical stirrups. The horizontal reinforcing was the same as before. The spacing and fastening of the inclined bars has been discussed in preceding paragraphs and need not be repeated here.

Holes were bored into the side of the beam in Fig. 16 (a) and (b) so that the extension of four of the eight inclined bars could be measured. The four nearest to the surface were exposed because the others could not be readily reached with the extensometer. The deformation in the horizontal bars also was measured in four

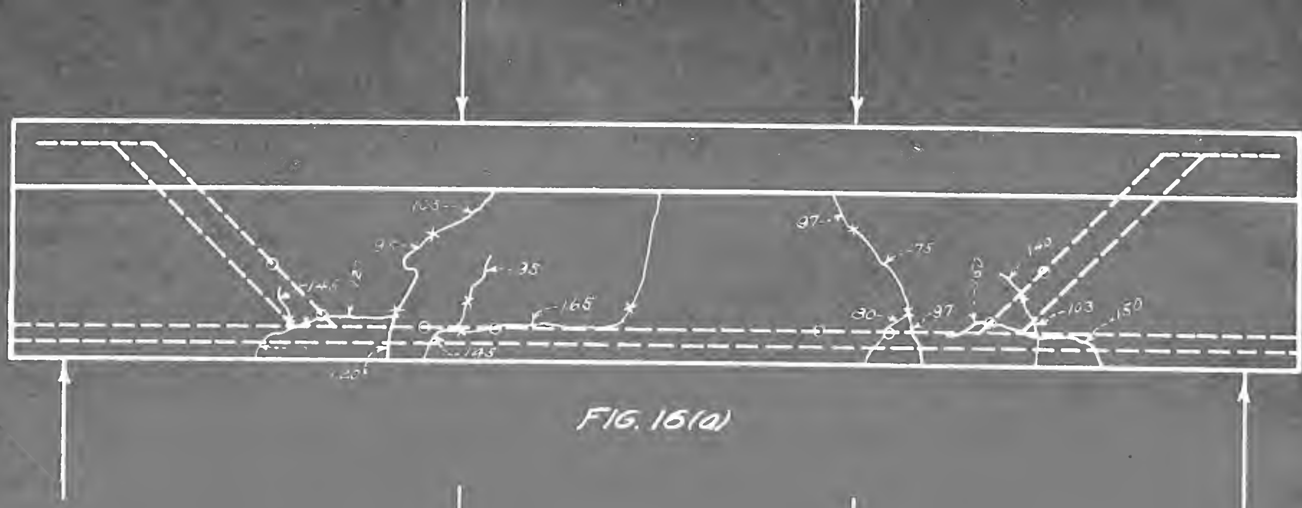


FIG. 16(a)

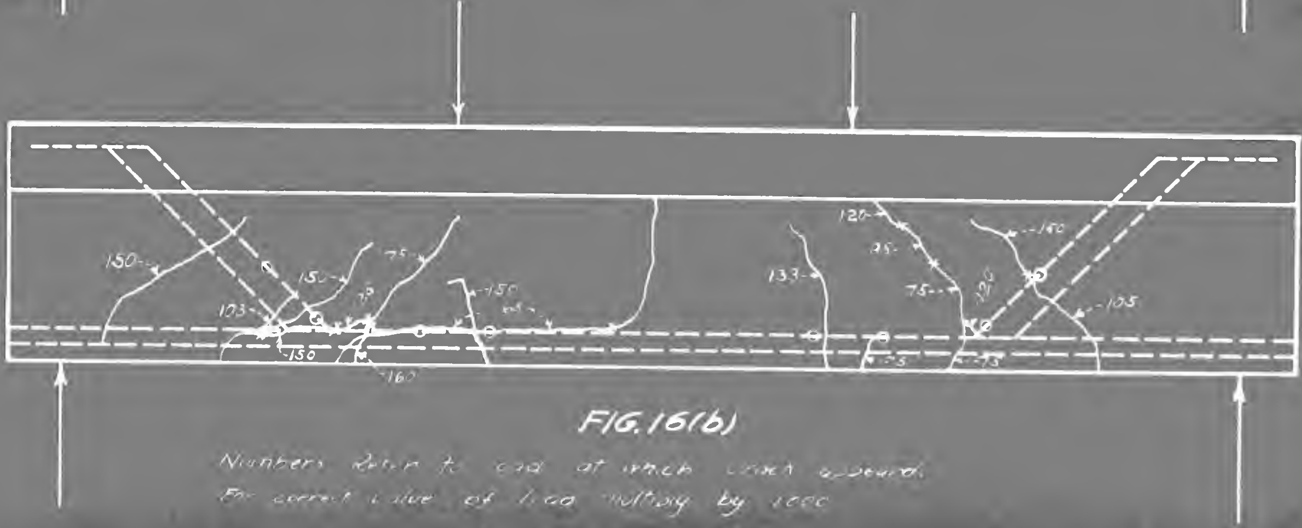


FIG. 16(b)

Numbers refer to load at which crack appeared.
For correct value of load multiply by 1000

places. Two holes were bored eight inches apart, four inches on each side of the load points, along the lower line of horizontal steel. This was done on both sides of the beam. The openings along the inclined bars were eight inches apart, and placed along the bar below the vertical axis of the beam, so that the full action of the inclined bar was obtained.

The dead weight of the concrete beam plus I beam and plates was five thousand two hundred ten pounds. At this reading of the scale of the machine the initial readings of the punch marks upon the steel were taken ith the extensometer.

The initial applied load was such that the scale reading of the machine was fifteen thousand pounds. Increments of five thousand and ten thousand pounds were taken until the value of one hundred sixty-five thousand pounds was reached. The

Beam No. 3. South side; failure occurred at 165,000 pounds.

North side of same beam; failure as above. Note openings at the elevation of the horizontal reinforcement.

concrete had become somewhat weakened so that the load began to drop, staying temporarily constant at one hundred forty-two thousand pounds and one hundred thirty-five thousand pounds.

The load was then slowly increased up to one hundred sixty-two thousand pounds from which value it again began to drop rapidly balancing at one hundred forty-eight thousand one hundred thirty-nine thousand pounds.

The load was increased to one hundred forty-nine thousand; again it dropped back to one hundred thirty-six thousand five hundred pounds. These various repetitions of load show that although beam had reached its ultimate strength that it by no means had lost all of its resistance.

The first fine cracks were noticed at seventy-five thousand pounds and the

sequence of the formation of cracks from there on is shown in Figs. 16 (a) and (b). These figures show that the beam was fairly designed as no sudden failure took place in any one side of the beam, but that it occurred at the same time uniformly throughout the beam.

The final failure took place not only in the diagonal tension crack, but also in an opening running along the line of horizontal reinforcement. This is the typical failure for a beam having a web reinforcement of inclined bars.

Warning of failure became obvious through the many full cracks formed in the web, on both sides of the beam, nearly at right angles to the slope of the inclined bars. The inclined bars serve very well in holding together the concrete after it has failed. When properly spaced they are more effective than stirrup reinforcement.

The actual stresses were calculated and compared in the tables with the theoretical stresses.

To calculate these stresses, stress deformation curves were plotted with the readings taken at the openings 1,2, 3 and 4 on both sides of the beam.

The tangents of these curves were extended back to the abscissa corresponding to the zero value of ordinates. This gives a new zero value for abscissa, giving deflections which would occur in the steel if it were not constrained by the concrete. Using these new values of deflections the stresses were calculated by means of the formula $S = E_x$ unit elongation. The theoretical stresses in the inclined bars were calculated by the formula

$$f_s = \frac{.707 \times V_s}{jd\ A_s}$$

which in this case becomes equal to

$$\frac{.707 \times 5 \times V}{.9 \times 23.75 \times 1.76} = 0.094\ V.$$

The results tabulated show that the inclined bars more nearly take the theoretical stresses than the vertical stirrups. The actual stresses in the horizontal bars seem to be on an average only about 0.60 as great as those calculated from the bending moment formula.

Formulas for calculating the stresses in a honogeneous materials show approximately what stresses actually exist. In structures such as the above beam, and those reinforced with stirrups formulas cannot give such values because they vary so much.

Beam No. 3 after failure at a load of 15[illegible],[illegible] pounds.

TABLE V

72 Day Test of Beam No. 3.

Numbers at head of columns refer to openings in beam.

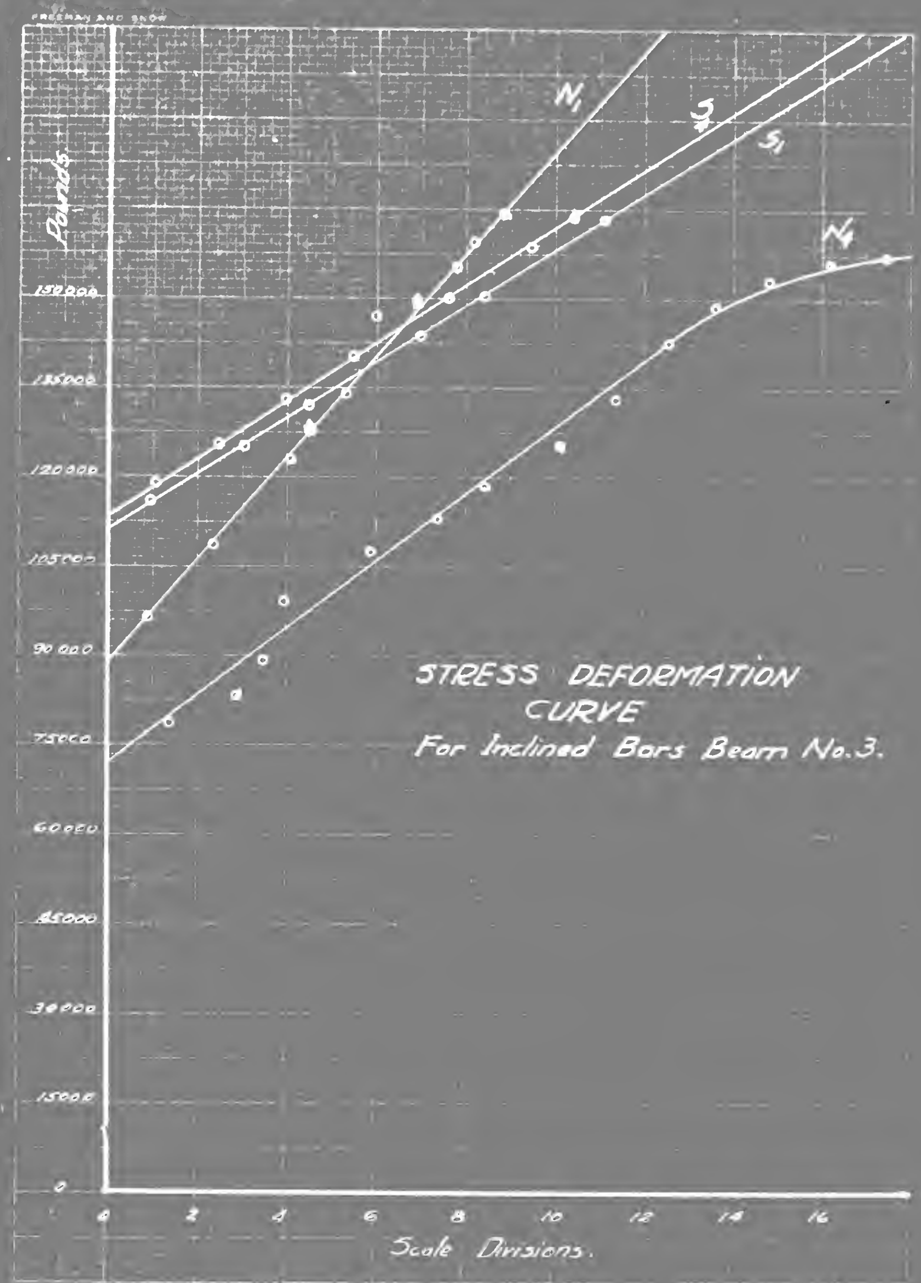
NORTH

Load	1	2	3	4
5,510	25.5	16.0	15.8	7.3
15,000	25.2	17.0	12.0	4.2
20,000	26.5	22.0	12.2	8.2
25,000	26.0	17.5	15.0	7.5
30,000	26.0	17.0	15.0	7.7
35,000	26.0	17.0	15.0	6.5
45,000	26.0	18.0	14.8	7.0
55,000	25.8	18.0	15.0	7.5
65,000	26.0	18.0	13.0	7.0
75,000	26.0	16.0	10.5	7.0
80,000	26.0	15.0	9.0	6.0
85,000	26.0	15.0	8.0	4.5
90,000	26.0	14.5	7.5	4.0
95,000	26.0	14.5	6.5	4.0
100,000	26.0	12.5	6.3	3.5
105,000	26.0	11.5	6.5	2.5
110,000	26.0	11.0	5.5	1.5
115,000	26.0	11.0	3.5	0.0
120,000	25.0	9.0	4.0	-99.0
125,000	23.0	9.0	3.2	97.5
135,000	21.5	8.5	1.5	96.0
140,000	21.5	8.0	0.0	96.0
145,000	20.0	7.5	-99.0	95.0
150,000	19.0	7.5	98.0	94.0
155,000	18.0	7.0	99.0	92.5
160,000	16.5	7.0	99.0	89.0
(a) 165,000	17.0	8.0	15.0	91.0
(b) 162,000	16.0	8.0	8.0	92.0
(c) 149,000	23	10.0	10.0	93.0
(d) 156,000	24	10.0	10.0	90.0

SOUTH

1	2	3	4
2.5	28.5	26.0	11.0
-99.0	28.5	23.0	13.0
98.0	24.5	23.7	18.0
97.0	28.0	24.0	18.0
97.5	24.0	24.0	18.5
97.5	24.0	24.0	18.0
97.5	24.0	24.0	18.0
97.5	24.0	23.5	18.0
97.5	22.0	22.0	18.0
97.5	20.8	20.5	18.0
97.5	19.0	18.0	18.0
97.5	18.0	17.5	18.0
97.5	17.0	17.5	18.0
97.5	16.5	16.0	17.0
97.5	14.0	14.0	17.5
97.5	11.5	12.0	17.5
97.0	11.5	10.0	17.5
97.5	11.0	11.0	16.0
96.0	10.5	9.5	14.0
95.0	10.0	9.0	14.5
93.5	10.5	8.5	14.0
92.5	10.0	9.0	12.5
91.5	9.0	8.0	11.0
91.0	9.0	7.0	10.5
90.0	7.5	6.5	10.0
90.0	8.5	4.0	10.0
89.0	9.0	7.0	8.0
88.0	10.0	8.0	8.0
93.0	15.0	15.0	10.0
		10.0	5.0

(a) Beam balanced at 142,000 lbs.
(b) Beam balanced at 148,000 lbs.
(c) Beam balanced at 136,500 lbs.
(d) Beam balanced at 146,000 lbs.


FREEMAN AND SNOW
Pounds
150000
135000
120000
105000
90000
75000
60000
45000
30000
15000
0
0
2
4
6
8
10
12
14
16
Scale Divisions.
N_1
S_4
S_1
N_4
STRESS DEFORMATION
CURVE
For Inclined Bars Beam No. 3.

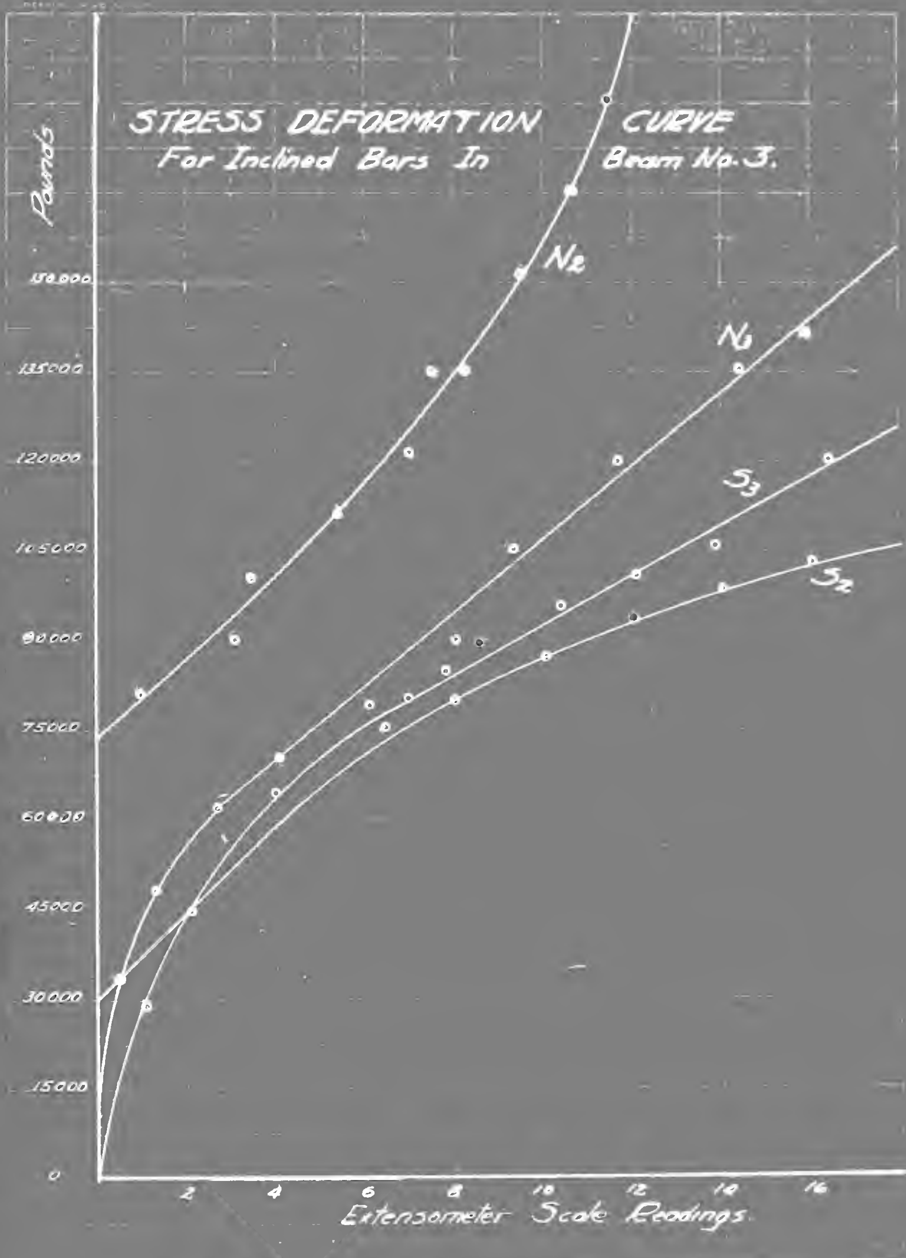

STRESS DEFORMATION CURVE
For Inclined Bars In Beam No. 3.
Pounds
150000
135000
120000
105000
90000
75000
60000
45000
30000
15000
0
2
4
6
8
10
12
14
16
Extensometer Scale Readings.
N2
N1
S3
S2

TABLE VI

Stresses in Inclined Bars.

All Stresses in pounds per square inch.

Load	Theoretical	Actual Stress in N.	Actual Stress N4	Actual Stress S1	Actual Stress S4
90,000	4,930		1,700	0	0
105,000	5,630	1,460	4,000	0	0
120,000	6,850	2,780	6,650	770	1,230
135,000	7,050	4,080	8,800	3,850	3,040
150,000	7,050	5,400	10,400	6,150	5,850
165,000	7,750	680	13,900	9,240	8,000

The deflections in Table V show that no notable tension is being developed by the inclined bars until about 90,000 pounds is reached; therefire calculations are shown from that value on.

TABLE VII

Stresses in Horizontal Bars.

Load	Theoretical	Actual N2	Actual N3	S3	S2
60,000	17,300	6,500	9.060	9,000	10,900
75,000	21,600	7,700	12,000	12,000	13,100
90,000	26,000	9,550	14,000	14,000	17,400
105,000	30,400	11,250	15,400	18,300	22,300
120,000	34,600	12,900	16,700	20,600	24,000
135,000	38,900	14,260	18,800	21,300	24,000
150,000	43,400	15,100	22,300	22,000	24,000
165,000	47,500	16,000	27,800	22,500	30,000

CONCLUSION.

The test of each beam was separately discussed and some conclusions noted. The results are as follows:

(1) That the actual stresses in the stirrups cannot be calculated by the formula $A_s f_s = \frac{2}{3}\frac{V_s}{jd}$, because the ratio of the stress in the stirrups to the total stress due to shear is much less than two thirds, the figures in the preceding work showing it to be between the values of one tenth and four tenths. These stresses, however, do not occur in stirrups until the concrete has very nearly failed. The load at which these particular beams were designed to fail by diagonal tension produced no stress in the stirrups.

However, when web reinforcement is designed according to the formula $A_s f_s = \frac{2}{3}\frac{V_s}{jd}$ a very large factor of safety

is introduced, in fact it is believed that this is too large to be economical.

Tests at various experiment stations show that beams reinforced by stirrups are stronger than those without them, so it can be seen that they are of some value, but no attempt should be made to calculate the stresses in them. Of course some standard way of placing the stirrups must be used so that engineers will not throw in reinforcement unsystematically. Since the above formula provides a large factor of safety it may be used as well as any other.

The tension in the stirrups occur after actual displacement in the concrete, due to the failure of it, takes place.

(2) In acting as stiffening agent in the beam the stirrups prevent sudden diagonal tension failures, as no sudden failure occurs as in beams devoid of web

reinforcing. Fine cracks appear which serve as a warning. The greater the stiffness of the beam, the greater its resistance to vertical shear will be.

(3) Repeated loading shows that when diagonal tension failure takes place in the concrete. the stirrups will take stress from the beginning of the second application of load.

(4) The extensometer readings show a shortening in the stirrups followed by an elongation which begins at about the first signs of fine cracks in the concrete approximately near the stirrups.

It must be remembered whereas concrete and steel act as homogeneous materials in some respects, their mechanical action, and the stresses set up by it cannot be very well predicted, as all depends upon the various conditions of construction, such as storage, temperature, type of

cement, intimacy of mixture, amount of water, rodding and t amping, and many others. The bond between the steel and concrete is effected by these; poor bond may be the cause of failure where everything else may be as designed.

The authors consider this merely as the beginning of an investigation of the actual conditions in the webs of reinforced concrete beams. It is hoped that in the future a more extensive research along this line will be made in which the results may be taken from the observation of many beams. It is suggested that if such tests are made, one beam without web reinforcing should be made for each one containing it. Then more definite deduction in regard to the steel will be possible.

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